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Ref: 8EPR-SR January 31, 2011

MEMORANDUM

SUBJECT: Libby Asbestos Site, Operable Unit 3
Section 5.0 of the Phase III Sampling and Analysis Plan
Proposed Geotechnical Investigations

FROM: Bonnie Lavelle
Remedial Project Manager
Libby Asbestos Site, OU3

TO: Site File

On January 26, 2009, EPA transmitted the draft Phase III Sampling and Analysis Plan for Operable Unit 3 of the Libby Asbestos Superfund Site (Phase III SAP) to external reviewers. As a result of extensive discussions with Remedium during a meeting on February 5, 2009, EPA decided to revise Section 5.0 of the Phase III SAP that described geotechnical investigations to support the remedial investigation/feasibility study. On May 26, 2009 EPA issued the final Phase III SAP with a placeholder for Section 5.0 with the intention of inserting this section once it was finalized by EPA.

The attached documents provide the record of EPA's revisions to Section 5.0 of the Phase III SAP, the Geotechnical Investigation. The documents are:

- **Attachment 1:** First revision, submitted to EPA in April 2009 by NewFields, an EPA contactor, with EPA internal review comments.
- **Attachment 2:** Additional revision of Section 5.3, Data Quality Objectives, submitted to EPA by NewFields on May 25, 2009.
- **Attachment 3:** Second revision, submitted to EPA in July 2010 by Formation Environmental (formerly NewFields).
- **Attachment 4:** EPA internal review comments on second revision.
- **Attachment 5:** Revised draft of Section 5.4, Sampling Design, transmitted by EPA to Remedium on December 16, 2010.

ATTACHMENT 1

INCLUDING THE NO ACTION ALTERNATIVE

5.0 OTHER DATA NEEDS FOR RI/FS

Additional geotechnical data are needed to support characterization of site conditions and evaluation of remedial alternatives in the FS. The long term effectiveness of the No Action alternative will require information to assess the stability of mine features and their potential to release materials into the environment. In addition, depending on the findings of the environmental sampling and human health and ecological risk assessments, evaluation of a range of source control alternatives ~~is anticipated~~ in the FS. Potential source areas to be investigated IN THE PHASE III PROGRAM are identified as follows:

MAY BE NEEDED

OVERALL PROTECTION OF HUMAN HEALTH AND THE ENVIRONMENT AND

CONTAMINATED

- Tailing Storage Facility;
- Coarse Tailings Pile;
- Surface Mine Area; and
- Waste Rock Piles.

This section addresses the data requirements, data quality assessment OF EXISTING DATA, data quality objectives, sampling design, analytical requirements and quality control that are needed for the required geotechnical data at OU3.

5.1 Data Requirements

This section presents the background information necessary to assess the geotechnical engineering data requirements for OU3 RI/FS. This information is developed from various sources.

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Tailing Storage Facility

CONTAINS FINE TAILING FROM THE PROCESSING OF VERMICULITE. THE TAILINGS ARE

The Tailing Storage Facility on Rainy Creek is impounded by a high-hazard, 135-feet high dam (127 ft reported by Harding, Lawson and Associates [HLA], 1992), initially constructed in 1971 with a 50-feet high starter dam (Schafer and Assoc., 1992). The dam is classified as high hazard due to its size and presence of hazardous constituents. The tailing dam is also known more recently as the Kootenai Development Impoundment Dam (Billmayer and Remedium, 2007) and previously as the W.R. Grace Vermiculite Tailings Impoundment or the W.R. Grace Dam, Rainy Creek, Montana (Schafer and Associates, 1992 and HLA, 1992). Most recently the tailing dam has been called the Kootenai Development Impoundment Dam (KDID) in the 2008-2009 periodic Owners' inspection report (Billmayer & Hafferman, 2009)

The original tailing dam designer was HLA, which performed the design in 1971. Several drawings were reviewed from this design including design drawings from 1979 (W.R. Grace Co., 1979). The original design drawings indicate a 50-feet high starter dam with 2:1 side slopes, a 40-feet wide crest and a downstream chimney drain. The drainage system is shown starting upstream from the starter dam and extending along the foundation through each dam

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raise. Perforated cross drains are shown in the foundation for the Phase 1 and Phase 3 downstream embankment raises. Initial embankment materials are shown as "Zone 3 and abutment excavation" material, although no further description of these materials is given to identify if they are silty gravels, sands or clays etc. The drawings indicate three downstream raises of approximately 10 to 25 feet (Phases 1, 3 and 5) and two smaller centerline crest raises of approximately 5 to 10 feet (Phases 2 and 4). The fourth centerline raise to El. 2900 ft AMSL occurred in 1979 (Shafer and Associates, 1992). The fifth raise to El. 2926 ft AMSL is shown as a downstream raise apparently performed in 1981 (Billmayer & Hafferman, 2009). The downstream slope is shown as 2:1 with two benches each 10 feet wide. The centerline of the starter dam is shown as approximately 100-feet upstream from the 1979 dam crest centerline. The maximum design height of the embankment appears to have been 200 feet with downstream raises (Billmayer & Hafferman, 2009). Therefore, the current embankment height is approximately 67 percent of the final intended design height.

* → OR WHERE THEY ORIGINATED

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$\frac{135'}{200'} = 67.5\%$

The crest length is approximately 1,100 feet, a concrete box culvert with principal spillway discharge is located on the left (east) abutment and an emergency spillway at a higher elevation is located on the right (west) abutment. The principal spillway has an outfall to Rainey Creek below the dam and the emergency spillway does not appear to have an outfall to the creek.

It appears from original drawings that foundation stripping up to about 5 feet in the valley bottom was performed to remove surface silts and that the abutments were stripped and benched. Original gravel blanket drains are shown in the design with perforated pipes to the downstream face, which were extended and added to during subsequent raises. Coarse tailing materials from the over-steepened area of the Coarse Tailing Pile were reportedly used in one or more of the dam raises. However, it is not clear where the coarse tailings might have been used in the embankment. Materials used for each embankment raise, whether centerline or downstream, are not defined in the available drawings.

WHERE WAS THIS REPORTED?

Deleted: This will require further definition and field verification prior to a final remedial design.

A series of eight boreholes to maximum depths of about 55 feet below original ground surface and 4 test pits to a maximum of about 17 feet below ground surface (ft bgs) are shown on the 1971 drawings in the vicinity of the starter dam and downstream of the starter dam. The borings do not have SPT values and do not indicate consistency of materials (loose, dense, very dense etc.). Silt depths of up to 5 feet are indicated underlain by gravelly sand to sandy gravel of unknown consistency. The pyroxenite bedrock underneath the dam appears to be approximately 26-36 ft bgs. Bedrock on the right side near the west abutment appears to be deeper, about 40 to 45 ft bgs. Bedrock on the right abutment appears to be about 12 to 18 ft bgs and on the left abutment appears to vary from about 8 to 12 ft bgs. The rock is only about 1 ft bgs further up the left abutment area. A zone of silts and clays is indicated at depths of approximately 19 to 26 ft bgs and 35 to 37 ft bgs near the right abutment downstream toe. It is not indicated if these are soft zones. No test pits or borings are indicated in the impoundment area.

SHOULD BE 14

The surface discharge from the tailing impoundment discharges through a reinforced concrete principal spillway at a crest elevation of 2,897 ft AMSL located on the left (east) abutment. An inlet channel presently extends from the pond several hundred feet upstream from the dam crest

29' LOWER THAN ℄ CREST ELEVATION

to the principal spillway inlet. The principal spillway consists of an 8-feet wide by 4-feet high concrete box culvert approximately 169-feet long and an 8-feet wide by 3-feet high concrete discharge channel approximately 965-feet long with a concrete and riprap outfall. The principal spillway has a reported full-channel discharge capacity of 731 cubic feet per second (cfs) with the water surface at the dam crest. The concrete structures are reported to be partially cracked with some rocks and debris near the inlet. → REFERENCE ?

The principal spillway and an emergency spillway, located on the right (east) abutment, are reportedly designed for one-half of the probable maximum flood (1/2-PMF; Schafer and Assoc., 1992). The peak inflow from the total Rainy Creek and Fleetwood Creek upstream drainage area (9.4 square miles) for the ½-PMF event was computed to be 5,838 cfs. The storage capacity of the impoundment was estimated to be approximately 1,302 acre-feet at the dam crest (Schafer and Assoc., 1992). Routing the ½-PMF flood hydrograph through the reservoir resulted in a peak discharge flow significantly lower than the peak inflow, and the present system of concrete principal spillway on one abutment and earth-riprap emergency spillway on the other abutment was recommended and constructed in the early 1990s. The emergency spillway is reported to be approximately 35-feet wide by 380-feet long with riprap erosion protection at an elevation of approximately 2,922 ft AMSL. The capacity of this emergency spillway with the water surface at the dam crest is reported to be 1,129 cfs (Billmayer & Hafferman, 2009). ~~Thus, the~~ combined discharge capacity of the principal and emergency spillways is approximately 1,860 cfs. — WHERE DID THIS VALUE COME FROM ?

BETTER EXPLAIN THIS

PRINCIPAL SPILLWAY: 731 CFS
EMERGENCY SPILLWAY: 1129
1860

41% CHANCE OF SOMEONE DYING

Recent risk-based analyses of the tailing dam concluded that the potential loss-of-life is 0.41 (Billmayer & Hafferman, 2009). Based on the current Montana spillway standards, the spillway design flow is therefore downgraded to an inflow design flood having a recurrence interval of 500 years. Analyses performed for this flood event determined the peak inflow from Rainey and Fleetwood Creeks to be 351 cfs utilizing USGS regression equations for Montana stream peak flows. This method provides an approximate method of determining peak flow with a standard error of prediction of approximately 67 to 79 percent. Therefore, based on this analysis, the existing peak design flow of 1,860 cfs for the spillway system, which is based on the ½-PMF inflow, is well in excess of the latest peak inflow from the 500-year flood event. Analyses were performed assuming loss of upstream vegetation due to a forest fire with a ground cover of approximately 20 percent. The peak inflows for this condition were estimated to be approximately 851 cfs. Environmental risk analyses have not been performed for the tailing storage facility.

PRINCIPAL SPILLWAY – 731 CFS
EMERGENCY SPILLWAY – 1129 CFS
1860 CFS

By comparison, previous studies performed using a hydrograph analysis with full forest vegetation conditions and an overall hydrologic Curve Number of 60, estimated the combined peak flow from the 100-year, 24-hour storm event in Rainy and Fleetwood Creeks to be approximately 460 cfs (Schafer and Assoc., 1992).

WHY DELETE THIS ?

Deleted: The differing peak flows from the two methodologies will need to be fully understood and reconciled for final FS analyses.

Several open-tube piezometers are located in and near the dam embankment, which indicate either dry conditions or relatively low water levels. The maximum phreatic surface is reported to

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HOW MANY ?

be approximately 94 feet below the crest of the dam, or approximately 40 feet above the base of the dam. One piezometer is reported to fluctuate several feet each year and up to a maximum of 30 feet. The peak of the highest phreatic water surface each year corresponds to the peak of the snowmelt/rain runoff in the area in the late spring. Only one piezometer is located within the tailing impoundment, which is reported to have not been measurable the last few dam inspections. This piezometer (P-O) consists of a 2-inch diameter PVC casing with two ¼-inch tubes inside, and appears to require compressed air for reading (Billmayer & Hafferman, 2009).

TOE DRAINS — I THINK THERE ARE 13

How Many?

A series of seepage-control pipes are located on the downstream embankment which have been maintained periodically (Billmayer, 2007a). The most recent dam inspections in December 2008 and January 2009 reported on each of the twelve seepage pipes exiting the downstream embankment (Billmayer & Hafferman, 2009). This report also discussed the various — How many ? 5? 14? piezometers in and near the dam embankment. It was concluded that the drains and the phreatic surface indicated by the piezometers follow the yearly surface water flow fluctuations. It was further concluded that the majority of the volume of stream flow upstream of the tailing impoundment infiltrates the tailings and subsequently reports to the toe drainage system. A portion of the surface flow also discharges through the principal spillway during the late spring most years.

SEISMIC CONSIDERATIONS

Previous studies have concluded that the tailing embankment is stable during static and seismic conditions with acceptable deformations reported for an analysis assuming a maximum credible earthquake producing a horizontal ground acceleration of 0.30g (HLA, 1992). These analyses were based on the state of Montana standards prior to development of the new Montana Dam Safety Standards for High-Hazard Dams. Previous analyses appear to have utilized two-dimensional models in 1992. Recent stability analyses, with updated seismicity conditions, do not appear to have been performed for the structure. Finite element analyses of stress conditions utilizing state-of-the-art models, do not appear to have been performed for the dam and foundation. *

SEEPAGE THROUGH DAM

Seepage through the dam has been identified as a potential long-term stability concern, particularly if the impounded water is adjacent to the dam. A levee was recommended in the 1992 HLA study to be located approximately 500 feet upstream from the dam crest to prevent the pond from reaching the dam; however, the levee was not constructed. *

UPDATE BASED ON NEW REPORTS FROM 2010

Tailings consist of interbedded layers of soft to stiff elastic silt (60%) and loose to medium dense poorly-graded sands and silty sand (40%) with mica and pyrite flakes. Based on two borings in the east side of the impoundment, the maximum thickness of tailings in the impoundment is approximately 70 to 75 feet (HLA, 1992). Confirmation of these depths and estimation of depth variations over the impoundment area, particularly further upstream, have not been performed. * The loose silty sand tailing materials are reported to have liquefaction potential during seismic events (HLA, 1992).

Embankment soils consist of dense to very dense, well graded silty sands. The overall downstream embankment slope is shown on stability models to be approximately 4(horizontal):1(vertical), although existing slopes appear to be steeper than this. The right abutment is underlain by a thick blanket of glacial outwash and till from a few feet to 40 feet thick. The left abutment slope is blanketed by a relatively thin mantle of slope debris and remnants of a lateral moraine near the base of the canyon slope with an intermediate 4-feet thick zone of highly permeable, relatively clean sand. Natural foundation soils consist primarily of dense to very dense poorly-graded gravels, dense to very dense poorly-graded sands and moderately hard, friable pyroxenite bedrock with abundant magnetite and pyrite (HLA, 1992).

The Tailing Storage Facility covers an area of approximately 53 acres (75 acres at maximum flood pool), a portion of which contains open water area of several acres depending upon the inflow to the impoundment. The volume of impounded water at the emergency spillway crest is approximately 937 acre-feet and the volume at the dam crest is approximately 1,302 acre-feet (Schafer and Assoc., 1992). The impounded water is typically approximately 500 feet upstream of the tailings dam; however, during extreme flood events water could be impounded adjacent to the dam. The impounded water discharged over the spillway during the 2008 spring runoff period, and typically discharges during normal precipitation years.

HOW IS THIS CALCULATED?

ALTERNATIVE?

The Draft Environmental Assessment for the site (Montana Department of State Lands, 1992) identified a number of concerns with a full diversion of Rainy Creek around the Tailing Storage Facility including the following: *"The full diversion alternate increases the potential for failure, and decreases the safety of the system---Stability of the structure in a massive flood condition would be problematic---The channels carrying the diverted flows would be very large, and inherently less stable than smaller channels, particularly when constructed in the side of a hill as they would be in this case. From a hydrologic and geotechnical standpoint, any channel, natural or constructed, located above the low point in a drainage is generally not considered to provide good long-term service...Should diversion channels become plugged, or the system fail for some other reason, the flood flows would quickly breach the diversions and enter the impoundment".* This opinion was reiterated in the 1992 Schafer Engineering Analysis of Flood Routing Alternatives report.

WHY IS THIS DISCUSSION HERE?

DOES THIS PROVIDE THE REASON?

Deleted: These conditions will need to be investigated to make FS assessments of alternatives involving full diversion of Rainey Creek flows around the impoundment.

IF OTHER RA ALTERNATIVES ARE EVALUATED IN THE FS, ADD'L GEOTECH. DATA MAY BE REQUIRED

Geotechnical data including boring logs and laboratory testing were developed for this tailings dam during the 1992 study. Long-term maintenance of this tailings dam, to be evaluated in the FS, will be based on existing and additional data related to the geotechnical characteristics, confirmation of depth and extent of the tailings in the impoundment.

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RE-WORD

Coarse Tailing Pile

The Coarse Tailing Pile is located on the hillside east of the tailing impoundment and covers an area of approximately 140 acres. Based on topographic mapping, the total height of the Coarse

THE FS FOR OU-3 WILL INCLUDE AN EVALUATION ~~&~~ OF THE LONG TERM EFFECTIVENESS OF THE TAILINGS DAM. ~~WILL BE REQUIRED IN THE FS.~~ SUCH AN EVALUATION WILL BE BASED ON EXISTING DATA. HOWEVER, ADDITIONAL INFORMATION ON THE GEOTECHNICAL CHARACTERISTICS OF ________ (WHAT?), CONFIRMATION OF DEPTH AND EXTENT OF TAILINGS IN THE IMPOUNDMENT IS NEEDED.

FLEETWOOD POND ?

Tailing Pile is approximately 700 feet and has side slopes of approximately 3:1 to 4:1. The Coarse Tailing Pile reportedly has had some reclamation procedures applied as discussed below. A small surface impoundment is located at the east toe of this pile covering an area of approximately 16,000 square feet (sf). Fleetwood Creek extends along the north toe of the Coarse Tailing Pile and storm flow events likely extend the floodplain over the toe of the Coarse Tailing Pile although specific hydrologic/hydraulic information was not identified for review.

DEFINE

A portion of the Coarse Tailing Pile appears to be at the slopes of 2:1 to 4:1 and a portion, approximately 65 acres, is reported to be too steep or over-steepened. This over-steepened portion was reportedly the borrow source for a tailing dam raise, although documentation of this activity has not been identified. The northwest portion of the Coarse Tailing Pile extends into the upstream portion of the tailing impoundment and may have stability concerns.

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* CAN YOU PROVIDE A REFERENCE ? IF NO DOCUMENTATION, HOW CAN WE SAY "REPORTEDLY" ?

The Coarse Tailing Pile has reportedly undergone reclamation work including run-on control, contouring for runoff control, seeding, and planting of trees (Ray, 1999). The existing reclamation work has not been reviewed as part of this data needs assessment. This reclamation work was, however, reviewed for bond release by the Montana Department of Environmental Quality (MDEQ, 1999a).

A portion of the Coarse Tailing Pile reportedly experienced snowmelt/rain runoff erosion in 2007. This area reportedly required an estimated 6,500 cubic yards (cy) of restoration fill from nearby waste rock and relocation of an under-road culvert which apparently caused the washout (Remedium, 2007). Information regarding implementation of this erosion restoration was not identified for review. *

Geotechnical data for the Coarse Tailing Pile were not identified, other than anecdotal descriptions. Various issues were raised following bond release in 1999 including comparable stability and utility of reclaimed areas and levels of asbestos on the surface of reclaimed areas and potential for continuing release (MDEQ, 1999b). It appears that insufficient data exist to adequately assess the long-term stability of the over-steepened area near the Coarse Tailing Pile or to analyze source control remedial alternatives in the FS. Such data will include geologic reconnaissance and settlement monuments in the over-steepened area followed by inclinometers if determined to be necessary based on the reconnaissance and settlement study. Geotechnical index parameters will need to be obtained from test pit samples in the coarse tailing area to determine engineering characteristics and the test pits will assist in determining the volume of the coarse tailing pile. *

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WHAT DOES THIS MEAN ?

* LET'S NOT SAY "IT APPEARS THAT..."

WHAT DO WE MEAN BY SOURCE CONTROL ALTERNATIVES ?

Surface Mine Area

The Surface Mine Area covers an area of approximately 270 acres at the top of "Vermiculite Mountain". The disturbed area of the Surface Mine Area is contiguous with the mine waste rock piles immediately to the south. The former mill area was located just west of the Surface Mine Area and all associated facilities have been removed.

The Surface Mine Area includes the former "Glory Hole" which covers an area of approximately 15 acres southeast of the former mill area and adjacent to the Waste Rock Pile area. This was reportedly filled with miscellaneous mine waste debris (typical Class II landfill material), then covered and seeded as part of reclamation (Ray, 1999). FROM WHERE ?

The Surface Mine Area was reportedly reclaimed in the 1990s including regrading, seeding and planting (Ray, 1999). This area was inspected for bond release in 1999 by the MDEQ. Issues remaining included levels of asbestos on reclaimed areas and water quality concerns related to potentially hazardous materials disposed in the Glory Hole and other areas (MDEQ, 1999b). FIND OUT FROM COURTNEY

A stockpile of soil removed from residential yards in Libby is located on the Surface Mine Area.

Deleted: The volume of this soil stockpile has not been determined with accuracy.

ID AS MW-1

One monitoring well was installed adjacent to the Glory Hole in 2000 (MW-1) and another monitoring well was installed near the toe of the old waste dump (MW-2; W.R. Grace & Co., 2000). A log of the first monitoring well near the Glory Hole indicates approximately 4 feet of rock fill over approximately 16 feet of vermiculite with weathered pyroxenite below this to a depth of approximately 82 feet below ground surface. Biotite pyroxenite bedrock with traces of tremolite and diopside were identified from a depth of 82 feet to the bottom of the borehole at a depth of approximately 250 feet below ground surface. Groundwater was found at a depth of approximately 242 feet below ground surface and produced approximately 1 to 2 gallons per minute. Groundwater was reportedly identified in MW-2, which was completed to approximately 90 feet below ground surface. Groundwater from MW-2 was reportedly high in arsenic and lead (MDEQ, 2000), although such data were not identified. The location and log of MW-2 was not identified. → CONFIRM: CHECK PHASE IIB SAP

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A geo-hydrologic study was performed for areas north and south of the mine area including approximately 100 boreholes to depths of approximately 170 feet below ground surface (Zinner, 1982). These boreholes were located in the area between Waste Rock Piles 2 and 3 south of the mine and in an area east of the coarse tailing pile north of the mine. The boreholes indicated overburden materials from near zero to a maximum of approximately 90 feet below ground surface. Vermiculite pyroxenite was found below the overburden in thicknesses varying from approximately 40 to 170 feet below ground surface. Biotite pyroxenite bedrock was found at depths varying from approximately 40 to 190 feet below ground surface. Groundwater in these boreholes varied from the ground surface with artesian conditions in the area between the waste rock piles to approximately 140 feet below ground surface. The twelve artesian boreholes produced approximately 1 to 2 gpm water flow with release of trapped gas. Two boreholes north of the mine produced water flows of up to approximately 20 gpm. It was theorized that *"the aquifer is probably the result of a permeable zone of sandy and gravelly till overlain by a less pervious till"* (Harding and Lawson, 1974). Zinner theorized that *"the artesian conditions are*

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WE HAVE THIS LOG

WHERE ?

thereby the results of the upper inclination of glacial deposits to the canyon head where recharge takes place" (Zinner, 1982). Confirmation of these theories has not been made, and the areal extent of such conditions has not been determined.

Two deep boreholes were drilled in the mine area: one was drilled to a depth of 900 feet through the 22nd mining level (Hole 130) and one was drilled to a depth of 970 feet north of the mine area (Hole 131). The first deep borehole in the mine area indicated approximately 10 feet of overburden with 15 feet of vermiculite underlain by biotite pyroxenite to the 900 foot depth. This deep borehole produced approximately 25 gpm at the 500-foot depth, approximately 350 to 500 gpm was produced from a depth of 700 feet and drilling was stopped at 900 feet as approximately 1,000 to 2,000 gpm were being discharged to the surface. The final water level was approximately 66 feet below ground surface indicating the water level was under piezometric conditions. Another deep borehole was reportedly drilled 200 feet from Hole 130 which was reported to be under artesian conditions discharging approximately 5 gpm (Zinner, 1982). The second deep borehole north of the mine (Hole 130) did not encounter strong water producing zones as did Hole 130, although approximately 25 gpm was reported at a depth of approximately 500 feet below ground surface. The location and logs of deep boreholes 130 and 131 were not provided in the Zinner report.

DO WE HAVE THESE NOW? FROM MINE?

Anecdotal information regarding an underground mine beneath the Libby Surface Mine Area has not been confirmed. Information regarding such underground workings was not identified during this investigation.

FROM WHOM?

No additional geotechnical data were identified for review from the Surface Mine Area and it appears that insufficient data exist to characterize site conditions and to support evaluation of remedial alternatives in the FS. Geotechnical data are required including test pits and sampling to determine the index characteristics of soils.

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WHAT DATA ARE NEEDED?

Mine Waste Rock Piles

The mine Waste Rock Piles are located south of the surface mine and cover a total area of approximately 230 acres. The toes of the Waste Rock Piles extend southwest to Carney Creek in some locations and the side slopes appear to be roughly at the angle of repose, and some contouring has reportedly been performed. The Waste Rock Piles consist of three major piles south and southeast of the former mill. For the purposes of this investigation, the larger Waste Rock Pile located to the south of the former mill site is designated WRP-1, the middle pile is designated WRP-2 and the southeast pile is designated WRP-3.

Topographic maps indicate that the largest WRP-1 has a total height in excess of 850 feet on the west side and has an overall slope of approximately 2:1 with haul roads and benches. Portions of the east side of WRP-1 and WPR-2 and WRP-2 have heights of approximately 150 to 200 feet at side slopes varying from 1.2: to 1.4:1. The existing hillside slopes vary from approximately

?

2.5:1 to 2.8:1. The topographic maps and aerial views (Google, 2008) indicate areas of gully erosion from the waste rock piles.

WHERE? SURFACE WATER SAMPLES? GW?

THERE IS A

A small waste debris area, covering approximately 3 to 4 acres ~~was located~~ southwest of the mill. It was reported that miscellaneous debris (including drums) from this smaller Waste Rock Pile was disposed in an excavated area southwest of the mill site approximately 800 feet east of the Lower Pond (Ray Engineering, 1995). Water samples were reportedly obtained during reclamation of the small waste debris area but were not identified for review. This 1995 report also indicated movement of mine waste on the hillside thought to be caused by seepage from a spring and local areas of impounded water.

A landfarm was reportedly developed for treatment of wastes from a leaking underground storage tank at or near the mill site. Information and data for this landfarm treatment facility were not identified for review.

EPA MAY HAVE INFO

The Waste Rock Piles are reported to have undergone reclamation activities in the 1990s similar to the Coarse Tailing Pile and Surface Mine Area although the degree of reclamation is not known. A landslide area at one of the Waste Rock Piles covering approximately 45 acres exposed an old landfill in the 1990s, which was apparently reclaimed and the landfill debris was relocated elsewhere. The MDEQ reported that the landslide area had dried out and appeared to have stabilized (MDEQ, 1999a). However, hillside springs may re-appear at various locations depending upon snowpack and other factors. Decomposing vermiculite is typically a very weak material and its presence within the waste rock piles would tend to weaken the overall structures, particularly over time.

As discussed above, the Zinner report indicates artesian conditions in several of the boreholes between WRP-2 and WRP-3. It is not known with certainty how this artesian groundwater condition affects the stability of the waste rock piles. The Zinner report observed that the *"load created by the waste dumps and their impedance of water flow has created instability in the surrounding slopes and in the valley bottom"* (Zinner, 1982).

A 1992 environmental assessment determined that *"the waste rock dump has inherent stability problems due to the structure of the ore and waste rock. The dump is currently standing at the angle of repose (1.25 to 1.5:1)....As a result of mass wasting, the waste rock dump toe has encroached on the Carney Creek stream channel. The slumping of waste rock has forced the creek to cut a new channel through the waste rock that has rolled to the bottom of the drainage in the end dumping process used to form the waste rock dump"* (Montana Department of State Lands, 1992).

Long-term stability of the Waste Rock Piles will need to be evaluated in the FS under the No Action alternative, as well as for source control alternatives. Geotechnical data for the Waste Rock Piles were not identified and it appears there are insufficient data to assess the long-term

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IF SOURCE CONTROL IS DETERMINED TO BE NECESSARY AND IS A RAO, ALTERNATIVES . . . .

stability of the facilities or to analyze alternatives for a FS. Such data needed for analysis will include bulk samples for index parameters, compaction characteristics and strength parameters. Investigations will include test pits and geotechnical borings.

5.2 Data Quality Assessment

This data quality assessment includes a review of the identified engineering data for the Tailing Storage Facility, which primarily includes data for the impoundment dam related to stability and safety, and for the Surface Mine Area, which includes limited monitoring well data. Limited engineering data quality assessment is included for the Coarse Tailing Pile and the Waste Rock Pile areas based on very limited data adjacent to the areas.

<u>Tailing Storage Facility</u>

REFERENCES?

HOW OFTEN?, ANNUAL?

Because of the high-hazard rating of the tailing impoundment dam, geotechnical stability and hydrologic reports were completed for the facility in 1992 and periodic safety inspections have been performed since that time. Periodic safety inspections have found the structure to be safe with the implementation of additional maintenance measures associated with the downstream drainage system and with the addition of a reinforced concrete box culvert outlet through the left abutment and concrete discharge flume and chute downstream of the dam.

The geotechnical report completed in 1992 included 10 geotechnical borings to depths ranging from approximately 22.5 to 77 feet below ground surface (ft bgs). The soils were classified in accordance with American Society of Testing and Materials (ASTM) Standard D-2487 and visual-manual procedures were performed in accordance with ASTM D-2488. Standard Penetration Tests (SPTs) were performed in the borings in accordance with ASTM D-1586. Selected disturbed and undisturbed soil samples were tested for moisture content, dry density, Atterberg Limits, gradation, percent passing the No. 200 sieve, unconsolidated-undrained triaxial shear strength, consolidation and compaction characteristics. Although the testing procedures were not reviewed in detail, they were reportedly performed in accordance with established ASTM procedures. (REFERENCE)

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The original design in 1971 included 8 geotechnical borings and 14 test pits in the vicinity of the starter dam and downstream of the proposed dam embankment. No explorations were performed upstream in the impoundment area. The borings determined the depth to bedrock and the test pits indicated near surface conditions. Standard penetration data were not reported for the borings and the general subsurface conditions were described from the boring and test pits logs presented on the design drawings. It is not known what quality control procedures were utilized the sampling and analysis of subsurface materials.

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Fourteen piezometers at the tailing dam have been monitored during the periodic safety inspections. All but one of these piezometers was monitored in the 2007 and 2008-2009 inspection reports (Billmayer & Hafferman, 2009). The piezometer not measured is apparently located in the impoundment area approximately 300 feet northeast of the dam crest. Annual monitoring of the piezometers have reportedly found the phreatic surface in the dam to be relatively low, with a maximum height of approximately 3 to 4 feet above the dam foundation (HLA, 1992 and Billmayer, 2007a). Seven of the thirteen piezometers monitored contained water during the 2007 annual inspection and the latest inspection reported similar conditions.

— PIEZOMETERS ARE NOW MEASURED MONTHLY — REQUIRED BY DNRC

The 2007 inspection report concluded that the dam was in good to excellent condition and that no significant structural or maintenance concerns were found that would require immediate action (Billmayer, 2007a). The emergency action plan, operational plan, routine maintenance plan and piezometer monitoring logs were reported to be up-to-date and effectively addressed the structure and its components. The annual dam safety inspections have reportedly been approved by the Dam Safety Program of the Montana Department of Natural Resources (DNRC).

The 2007 dam inspection report recommended cleaning the seepage outlet drains and performing minor maintenance work on the dam and concrete box culvert and chute spillway, some of which was described in a Montana 310 permit application (Billmayer, 2007b). Some of this work has apparently been performed and recent photographs of the inside of some drain pipes indicate some corrosion and deterioration (Billmayer & Hafferman, 2009). Long-term effectiveness of the existing dam drainage system has not been performed [EVALUATED] and is suspect due to the corrosion of some drain pipes.

— CONSIDER THE ANNUAL INSPECTION & MAINTENANCE REQ'S — DO THESE PROVIDE LONG TERM EFFECTIVENESS?

The 2007 dam inspection report also recommended that a review of bank stability and seismic stability be performed (Billmayer, 2007a). Documentation of this review has not been identified. The 2007 inspection report also recommended that preparation for the 5-year operational permit renewal inspection be conducted no later than the fall of 2008. These recommendations included: 1) development of a complete catalog of all available documentation and reports for the tailing dam, 2) a complete review of the stability analysis based on the latest piezometer data, and 3) a review of the seismic stability of the embankment based on the new Montana Dam Safety Seismic standards for high-hazard dams in Montana.

SO, HAS THE SEISMIC STABILITY REVIEW BEEN DONE — IS IT IN 5 YR REVIEW REPORT?

Recent stability assessments have relied on previous geotechnical field investigations, laboratory analyses of materials and stability analysis models. The most recent inspection report (Billmayer & Hafferman, 2009) included a review of the 1992 seismic stability study by Harding Lawson. However, a critical review of updated seismic information was not apparently performed for the dam; the latest report stated agreement with the previous analyses performed in 1992. Additional stability analyses using recent state-of-the-art two-dimensional models have not been performed for the structure nor have finite element analyses of the dam structure stress conditions been performed.

Deleted: Such state-of-the-art analyses would be required for a final remedial design of an FS alternative for a major tailing storage facility impounding water.

NEED TO EVALUATE OVERALL PROTECTION OF PH & E AND LONG TERM EFFECTIVENESS OF NO ACTION

Data regarding embankment movement over time has not been identified. There do not appear to be any surveyed settlement monuments on the dam crest; only visual assessments of embankment movement and erosion ~~have been performed.~~ 4

IN FS

Some discrepancies exist regarding previous hydrologic analyses performed for the Tailing Storage Facility (Schafer and Assoc., 1992) and recent hydrologic analyses of inflow design floods (Billmayer & Hafferman, 2009). The USGS regression equation methodology utilized in recent analyses likely does not have the accuracy required (67-79% std. error) for a structure such as the Tailing Storage Facility Dam at the Libby Mine.

DISCUSS w/ BILLMAYER & HAFFERMAN ?

ACCEPTABLE UNDER FEDERAL DAM SAFETY REGS

Surface Mine Area

A few boreholes are reported to have been performed in the Surface Mine Area including some deep boreholes, although engineering-geologic data were not identified for the boreholes. One geologic log was identified for the monitoring well adjacent to the Glory Hole (MW-1) in the Surface Mine Area. It is not known what procedures were utilized in measurement of groundwater levels and what quality control procedures, if any, were utilized in the sampling and analysis of groundwater from the monitoring wells. Groundwater level data and water quality data were apparently not performed at the monitoring wells beyond the initial time period following installation of the wells. Groundwater well sampling has been performed as part of the RI. The log and location of reported MW-2 was not identified for review.

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Data and information regarding reported underground mine workings and how such workings may affect the surface mine area, or other site areas, have not been identified.

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Coarse Tailing Pile and Waste Rock Piles

As mentioned above, no geotechnical engineering data were identified for the Coarse Tailing Pile or the Waste Rock Pile areas other than anecdotal information regarding grain size of the coarse tailing materials. A geo-hydrologic report performed in the early 1980s (Zinner, 1981) presented general subsurface logs for areas east of the Coarse Tailing Pile, north of the surface mine, and between the eastern two waste rock piles. These indicated varying groundwater levels and some artesian conditions between the waste rock piles but did not define the lateral extent of such conditions. It is not known what quality control procedures were utilized in development of the wells, in measurement of the groundwater levels or in characterization of subsurface materials.

5.3 Data Quality Objectives

Data quality objectives (DQOs) define the type, quality, purpose and intended uses of data to be collected (EPA, 2006). The various steps involved in the DQO process will be followed to provide an effective project plan and to provide sufficient information to support key decisions regarding remedial alternatives. Such steps include: 1) State the problem that the study is designed to address, 2) Identify the decisions to be made with the data obtained, 3) Identify the types of data inputs needed to make the decision, 4) Define the bounds (in space and time) of the study, 5) Define the decision rule which will be used to make decisions, 6) Define the acceptable limits on decision errors, and 7) Optimize the design using information identified in Steps 1-6.

Statement of Problem

Remedial alternatives (including No Action) to be identified and evaluated in the FS require a sufficient amount of engineering information to support the evaluation of implementability, effectiveness and cost. Various remaining questions need to be addressed for each of the areas to be evaluated in the FS including the Tailing Storage Facility, the Coarse Tailing Pile, the Surface Mine Area and the Waste Rock Pile Area.

PROTECTION OF HH & ENVIRON.
LONG TERM EFFECTIVENESS & PERMANENCE

Tailing Storage Facility:

Geotechnical data have been developed previously for the Tailing Storage Facility dam for stability and safety evaluations. Such data appear to be acceptable for defining the general safety of the dam along with regular inspections and maintenance procedures. However questions remain regarding the impoundment area upstream from the dam including depth of tailings at various locations and the piezometric conditions within the impoundment. Such information is required to evaluate the effectiveness of impoundment capping alternatives to determine the amount of material required for capping following consolidation and the stability of capping scenarios. The No Action alternative would need to more fully evaluate the liquefaction potential of the tailing materials, which would require further information regarding the extent and characteristics of the impoundment.

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STATE AS QUESTIONS

Visual assessments of the tailing dam movement and erosion characteristics have been made during periodic safety inspections, which provide qualitative information regarding the surface conditions. However, questions remain regarding quantification of dam movements over time, and addressing such questions would be necessary for FS evaluations of long-term stability in addition to piezometer data and visual assessments.

Analysis of remedial measures at the Tailing Storage Facility impounding water would require state-of-the-art finite element analyses of the embankment and foundation stress conditions considering updated seismic studies, and detailed seepage analyses through the impoundment embankment and foundation. Additional data for such analyses would not be required for FS evaluations involving capping and removal of the impounded water in the facility. However, a No Action alternative including a long-term impounded reservoir should include such evaluations, and therefore would require further definition to complete the evaluations. If a No Action alternative is selected, additional field and laboratory geotechnical investigations would be recommended beyond those described in this document.

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WHAT?

- VERIFICATION BORINGS
- LABORATORY TESTING

Analyses of FS alternatives involving diversion of Rainey Creek around the Tailing Storage Facility will require an assessment of the stability of such diversions as well as final hydrologic studies to determine peak design floods and to verify peak design flows. The long-term stability of a diversion dam and channel system would be critical to the long-term effectiveness of a diversion system. Various questions regarding stability of potential diversion dam and diversion channel locations will need to be addressed. These should be addressed through careful field visual assessments followed by limited field investigations as necessary.

TAKE ~~OUT~~

KEEP IN NOW BASED ON LATEST DATA

Coarse Tailing Pile:

Various questions regarding long-term closure scenarios of the Coarse Tailing Pile remain. These include the questions regarding long-term erosion from the pile and overall stability. Basic engineering data such as coarse tailing geotechnical index parameters are needed for the Coarse Tailing Pile to analyze potential remedial alternatives in the FS. Basic index parameters such as grain size analyses are needed to assess the stability and long-term erosion characteristics of the Coarse Tailing Pile. In addition, the existing volume of coarse tailings needs to be estimated for detailed analyses required by the FS. Test pits through coarse tailing materials to underlying native materials would be necessary at a few locations to develop this estimate.

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The over-steepened area of the Coarse Tailing Pile east of the tailing impoundment has not been adequately defined to evaluate remedial alternatives for an FS. Therefore, additional investigations such as a geologic reconnaissance followed by settlement monuments or possible borehole inclinometers, if movement is occurring, will be needed for evaluation of this area.

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Surface Mine Area:

Questions remain regarding the Surface Mine Area which will need to be addressed for the FS such as definition of existing conditions of the Surface Mine Area. Such conditions include areas of wastes and areas of uncovered materials subject to erosion. The volume of residential yard soils currently stored at the Surface Mine Area, which was removed from Libby during that remediation, needs to be evaluated. This will require ground survey of the stockpile to estimate the volume of materials which could be used for partial capping alternatives at the mine or waste rock pile areas. Basic geotechnical index parameters such as grain size analyses are also needed to evaluate long-term erosion characteristics of the surface mine area.

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Questions regarding the reported underground mine workings will need to be addressed during the FS. However, no field explorations associated with this are recommended at this time.

Waste Rock Piles:

Various questions remain regarding the constituents in the waste rock piles, the volume of materials, the stability of the waste rock piles including groundwater conditions and the impacts to adjacent land and drainages.

FS alternatives investigating the feasibility of water treatment of drainage from waste rock pile runoff and drainage/seepage through the waste rock piles will need to consider the variability in water quality. Any instability in the waste rock piles would likely result in the release of higher LA (and potentially other constituent) concentrations in the drainage water. Furthermore, significant instability of the large waste piles may potentially endanger any constructed facilities downstream. Therefore, it is critical to the FS analyses to assess the long-term stability of the waste rock piles and their impact on surrounding land and Carney Creek.

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The existing waste rock piles are marginally stable under static conditions. However, during seismic conditions, or with seepage from springs or high groundwater conditions, they would not likely be stable. An analysis of the long-term stability, and erosion potential, of the waste rock piles is needed to evaluate the no action and capping/stabilization alternatives for the FS. Basic geotechnical data such as grain size analyses and material characterization are necessary to evaluate stability and erosion potential. The volume of materials in the waste rock piles is also needed for FS evaluations. A higher percentage of decomposing vermiculite in a particular area would be a weakening and therefore destabilizing element of the waste piles. Therefore, it is necessary to evaluate the approximate percentage of vermiculite in the waste piles and to evaluate the decomposition potential of various materials in the waste piles. FS evaluations will need to assess the global stability of the waste rock piles and their long-term effects on surrounding land.

Previous investigations have identified high groundwater conditions and some artesian conditions near the waste rock piles, in particular between the two waste pile on the east side. The full extent of such conditions has not been defined in the vicinity of the waste rock piles. Such information will be critical to the evaluation of remedial alternatives. Furthermore, information regarding creep or continuing movement over time is not available for these large structures. Therefore, field investigations including near-surface test pits, borings and settlement/movement monuments are needed to determine these factors.

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<u>Identify the Decision</u>

The engineering data collected during the OU3 RI Phase III is intended to help EPA decide if and what remedial alternatives are feasible and necessary to protect human health and/or ecological receptors from unacceptable risks from asbestos and any other mining-related contaminants at the Tailing Storage Facility, Surface Mine Area, Coarse Tailing Pile, and Waste Rock Piles over the long term.

Identify Types of Data Needed

Engineering data needed for the various areas at OU3 include:

- Boring logs and test pits with associated logging in accordance with generally accepted ASTM standards and Cone Penetrometer Testing (CPT) in the tailing impoundment area;

- Subsurface soil sampling for bulk samples and relatively undisturbed samples;

- Geotechnical laboratory testing for index parameters such as grain size analysis and Atterberg Limits and strength/durability characteristics as necessary depending upon location of sampling;

- Installation of piezometers and groundwater monitoring wells for assessment of groundwater and phreatic surfaces through the various facilities;

- Geologic reconnaissance and field inspection of existing conditions is needed in some areas as a first step in evaluation of long-term stability;

- Installation of settlement monuments, or borehole inclinometers if determined to be necessary, at various locations to assess long-term embankment and waste pile/hillside stability concerns; and

- Survey data to determine the location and elevation of borings, test pits, monitoring wells, piezometers and settlement monuments or inclinometers and to verify existing slope conditions at the facilities.

Define Bounds of Study

The spatial bounds of the study include the total areas currently occupied by the Tailing Storage Facility, Coarse Tailing Pile, Surface Mine Area, and Waste Rock Piles at OU3.

The temporal bounds of the study will include one season of geotechnical sampling and monitoring new monitoring wells and settlement monuments, or borehole inclinometers, as applicable during a typical range of annual groundwater conditions.

Define the Decision Rule

The quality and results of engineering data from OU3 will not be used to determine if remedial action is necessary. However, used in combination with the decision rules for human and ecological risks and for potential environmental impacts, the decision to recommend a particular remedial action will be made.

Define Acceptable Limits on Decision Errors

Acceptable limits on decision errors for engineering data from OU3 will be based on established engineering principals, accepted ASTM standards and engineering judgment. Typically, if data are within reasonable limits for the type of material sampled and within the range of previous data for similar materials or previous data for the facilities, the data will be accepted.

Optimize the Design

The sampling design is based on the DQO process, the site characteristics and scale, and anticipated needs to support identification and evaluation of remedial alternatives in the FS process. Locations of investigation and sampling points may be varied somewhat in the field from the plan depending upon field conditions encountered.

5.4 Sampling Design

The sampling design includes various field geotechnical cone penetrometer tests, borings and test pits with associated logging, sampling and testing of soils, tailings and waste rock from the borings and test pits. The approximate location of the test pits and borings are shown on Figure 5-1 and the program is summarized in Table 5-1. Ranges of sample numbers are provided. The lower number indicates the minimum requirement. If the material is heterogeneous more samples than the minimum may be required based on field observation. Depending upon initial field investigations in various areas, additional geotechnical investigations may be necessary in addition to those indicated on Table 5-1. Such areas may include the potential diversion locations for Rainey Creek around the Tailing Storage Facility and the over-steepened area of the Coarse Tailing Pile.

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Additional field geologic reconnaissance and inspection of existing conditions of various areas will also be performed as will land surveying of various features.

The location and elevation of all borings and test pits will be determined using survey-grade global positioning system (GPS) equipment. This equipment should provide the state plane coordinates to the nearest tenth of a foot and should provide the elevations to the nearest tenth of a foot based on feet above mean sea level.

All excavated test pits and boring cores will be documented with digital photography as necessary for each of the sampling locations.

Tailing Storage Facility

MONTHLY NOW

Previous investigations at the Tailing Storage Facility included a total of 10 borings developed for the 1992 geotechnical stability investigation of the impoundment dam. A total of 14 piezometers are annually monitored for dam safety inspections, none of which are within impounded tailings upstream of the dam. The original tailing dam design also included a series of borings in the vicinity of the dam which identify bedrock.

HOW MANY ?

A total of three cone penetrometer tests (CPT) are proposed at the Tailing Storage Facility impoundment to verify the thickness and characteristics of the impounded tailing materials and subsurface conditions: at the upstream area (approximately 500 feet upstream of the embankment) where a levee was proposed in the 1992 report, one approximately 1,000 feet upstream from the dam and one approximately 2,000 feet upstream from the dam as shown on Figure 5-1. The location of these CPTs is approximate and may vary in the field depending upon accessibility.

Use of CPT methods should utilize low-ground-pressure equipment to access areas not possible with a conventional drill rig. This method does not extract samples of subsurface materials for laboratory testing, but rather utilizes electronic friction cone or piezocone equipment to record the penetration resistance of subsurface strata. This data presents a qualitative correlation to physical properties of materials present such as shear strength, bearing capacity, void ratios and pore pressures. Since data is continuously recorded, the depth, thickness and variation in the stratigraphy provide a complete profile of the materials encountered. The CPT data will be presented in standard format for each location with associated analyses of the data.

SOP WITH RECORDING SHEETS

The existing non-functional piezometer in the impoundment area (P-0) should be repaired as necessary to assess the piezometric conditions in that area. It is recommended that a vibrating wire piezometer be installed to monitor pore pressure changes in the tailing materials. Such instruments provide a better assessment of piezometric conditions than open-tube piezometers in fine-grained materials such as tailings. Vibrating wire piezometers will be stainless steel units with durable pressure transducers capable of measuring pore pressures from -50 to 1,000 kilopascals (kPa; 145 pounds per square inch, psi) with an accuracy of plus or minus 0.1% full range. The unit shall be hermetically-sealed with durable cables and data loggers as necessary. The piezometer will be adequately protected with locking steel casings and concrete collars as necessary.

At least one concrete settlement monument will be placed on the tailing dam crest at the maximum section and will be surveyed to establish baseline data. This will provide needed

SPECIFY 1? 2?

Answer

quantification of embankment movements to complement and verify visual assessments and piezometer readings during periodic inspections. This will be a 10-inch diameter by 48-inch deep concrete cylinder installed vertically with the top approximately 3 inches above the existing ground surface. It may be either cast-in-place or precast concrete constructed with concrete having a minimum 28-day compressive strength of at least 3,500 pounds per square inch (psi). The top surface will have an embedded brass survey marker and will be surveyed for horizontal and vertical control from existing benchmarks; to the nearest 0.01 ft. Subsequent surveyed readings should then be performed twice per year through the FS period and following final remedial action. A survey point on the existing concrete principal spillway structure should also be established with associated baseline data.

WHAT'S ACCEPTABLE?

Visual assessments of existing ground conditions along potential diversion dam and diversion channel alignments for Rainey Creek upstream and adjacent to the Tailing Storage Facility will need to be made as a first step. If determined to be necessary during visual assessments, various test pits may be excavated at the potential diversion dam and channel locations with associated logging and sampling for index parameters.

IS THIS ASSOC. w/ A CREEK DIVERSION ALTERNATIVE?

<u>Coarse Tailing Pile</u>

Geotechnical investigations for the Coarse Tailing Pile will require four test pits. Approximate locations of the test pits are shown on Figure 5-1. The location of the test pits may vary in the field depending upon accessibility.

The test pits will be excavated with a large backhoe (track-hoe) to depths of approximately 10 to 12 feet. Slopes of test pits will be laid back to provide safe conditions as required by OSHA. The test pits will be logged by an experienced geologist or geotechnical engineer. Bulk samples of coarse tailing materials and underlying materials will be obtained and relatively undisturbed hand-driven samples will be obtained as possible. The hand-driven samples will be collected in 2-inch diameter by 4-inch long brass or stainless steel tubes. Alternatively 3-inch diameter by 6-inch long brass or stainless steel tubes could also be used.

Two test pits should be excavated near the toe of the Coarse Tailing Pile: one approximately 100 to 200 feet west of the pond and another approximately 800 to 1,000 feet west of this. These should be excavated to the base of the coarse tailing. Another test pit should be excavated about mid-way up the Coarse Tailing Pile slope in a relatively stable area and another should be excavated near the top of the Coarse Tailing Pile.

Bulk samples of cover soils, coarse tailing and subsurface materials should be collected from the test pits, as applicable. These samples should be tested for index properties including grain size analyses and Atterberg Limits as necessary depending amount of fines in the sample. In general, if the sample contains less than 10 percent fines (silt and clay passing the No. 200 sieve), Atterberg Limits will not be required, and the grain size analyses only need to be on the plus 200 sieve sizes. It is estimated that approximately 4 or 5 index property tests will be required, and that 4 moisture density tests will be performed on relatively undisturbed tube samples. In addition, approximately 3 or 4 samples of existing cover soils should be tested for organic content. An assessment of the areal extent and thickness of existing cover soils will be made for the entire Coarse Tailing Pile Area.

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A geologic reconnaissance will be performed in the over-steepened area of the Coarse Tailing Pile as a first step. This reconnaissance should evaluate all surface conditions including visible surface features, seeps, if any, and evidence of movement with associated digital photographic documentation. A land survey should be performed of the over-steepened area including the adjoining land on both sides, above and below the area. If determined to be necessary following the initial investigations, settlement monuments will be installed at selected locations to monitor movement of the area over time. If movement of the over-steepened area is occurring, inclinometer(s) may be installed to further evaluate movements at depth.

Surface Mine Area

The Surface Mine Area will be investigated with a series of test pits as shown on Figure 5-1. Four test pits are recommended in the Surface Mine Area with associated logging and sampling of cover soils, mine wastes and subsurface materials. The thickness of cover soils should be recorded at each location and the soil horizon should be logged as necessary.

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ASK USGS — HAVE THEY DONE WORK TO SUPPORT ESTIMATES OF AREAS OF UNCOVERED LA SUBJECT TO EROSION, AREAS OF COVER MATERIAL & THICKNESS

Bulk samples of surface soils and subsurface materials should be obtained for index testing: grain size analyses and Atterberg Limits, and for in-situ moisture density, as necessary. It is estimated that approximately 4 to 5 samples will be obtained for testing index parameters and that 4 relatively undisturbed tube samples will be tested for moisture-density. Additionally, cover soils should be tested for organic content. It is estimated that approximately 3 to 4 samples will be tested for organic content. An assessment of the areal extent and thickness of existing cover soils will be made in the Surface Mine Area.

The existing stockpile of residential soils will be surveyed to obtain an accurate volume of such materials.

Existing groundwater monitoring wells at the Site are being sampled as part of the RI. Data from this sampling will be used in the assessment of conditions in the Surface Mine Area and Waste Rock Pile Area.

Deleted: The two previous monitoring wells (MW-1 north of the Glory Hole and

Deleted: MW-2 near an old waste landfill) will be identified and monitored for groundwater levels and [sampled for water quality parameters] <<*Andy*

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Mine Waste Rock Piles

The three Waste Rock Piles will be investigated through a series of four test pits and three borings with two monitoring wells. The five test pits will include two or three on WRP-1 and

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one each on WRP-2 and WRP-3. Two or three of the test pits will be excavated near the top of the Waste Rock Piles and the remainder will be excavated in lower, accessible portions of the Waste Rock Piles.

One boring is proposed at the top of WRP-1 to assess the thickness of mine waste and subsurface soil horizon for stability. These borings should extend at least 5 feet into the native materials beneath the Waste Rock Pile for confirmation purposes. One boring each will be advanced through WRP-2 and WRP-3 within a few hundred feet of the previous borings which indicated artesian groundwater conditions. These should be located up-gradient and down-gradient of the previous boreholes performed in the Zinner Study Area 1. The exact locations will be field selected based on accessibility. Approximate locations of borings, monitoring wells and test pits shown on Figure 5-1 may vary in the field depending upon accessibility.

Two of the borings, in the WRP-2 and WRP-3 areas, will be developed as monitoring wells with 5 to 10 feet screened intervals within the groundwater zones encountered. It is anticipated that this will require 2-inch diameter Schedule 80 PVC casing. The MWs should be developed as necessary and monitored at least quarterly during the FS evaluation period. These monitoring wells should have protected steel pipe sections above ground surface with locking tops and concrete slabs at ground surface.

WHY ?

Three settlement monuments will be installed in the WRP areas to assess movement of these structures over time. These will be 10-inch diameter by 48-inch deep concrete cylinders installed vertically with the top approximately 3 inches above the existing ground surface. These may be either cast-in-place or precast concrete constructed with concrete having a minimum 28-day compressive strength of at least 3,500 pounds per square inch (psi). They will have brass survey markers embedded in the top and will be surveyed for horizontal and vertical control from existing benchmarks, to the nearest 0.01 ft.

Bulk samples of cover soils, waste rock and subsurface materials, as applicable should be obtained and tested for index parameters of grain size and Atterberg Limits, compaction and organic content of cover soils as necessary. It is estimated that approximately 10 to 12 index tests will be required, that approximately 3 or 4 organic content tests and that 3 or 4 compaction tests will be required. The size of bulk samples may vary from large zip-lock plastic bags for index and organic content tests to 5-gallon bucket samples for compaction tests. An assessment will be made of the approximate volume of vermiculite in the Waste Piles based on visual assessments and sampling of borings and test pits.

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Relatively undisturbed samples from borings or test pits will also be tested for in-situ moisture density. These in-situ moisture density tests will provide a definition of existing material conditions throughout the waste rock piles and some will be compared to the compaction tests to estimate the existing degree of compaction of materials. In addition, samples will be tested for strength to assess short and long-term stability of the Waste Rock Piles. The decomposition

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potential of materials within the waste rock piles will be evaluated through the use of freeze-thaw or slake-durability tests of selected samples of materials. It is estimated that 2 freeze-thaw tests and 2 slake-durability tests will be performed.

Data from existing monitoring wells MW-1, MW-2 and previous well information from the Zinner Report, in addition to new monitoring wells to be installed will be utilized to gain a better understanding of the geo-hydrologic conditions in the Surface Mine-Waste Rock Pile-Carney Creek area. Conceptualization and characterization of the groundwater system in the project area will be performed in accordance with accepted standards.

5.5 Analytical Requirements

The latest revision of the ASTM standards should be followed for all geotechnical soil and rock sampling and testing procedures. The following ASTM standards will be followed in sampling and analysis of geotechnical samples from OU3:

- Geotechnical Field Work should be performed in accordance with ASTM D-420 (Site Characterization for Engineering Design and Construction Purposes).

- Geologic reconnaissance procedures should be performed in accordance with standard ASTM procedures (Part 4.5 of ASTM D420-2003).

- Subsurface soils encountered in test pits and borings should be logged by an experienced geologist or geotechnical engineer in accordance with ASTM D-2487 (Classification of Soils for Engineering Purposes; Unified Soil Classification System) based on visual-manual procedures specified in ASTM D-2488 (Description and Identification of Soils; Visual-Manual Procedure).

- Standard penetration tests during boring shall be performed in accordance with ASTM D-1586 (Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils).

- Cone penetrometer testing shall be performed in accordance with ASTM D-5778 (Standard Test Method for Performing Friction Cone and Piezocone Penetration Testing of Soils).

- Relatively undisturbed cohesive soil and tailings samples should be obtained using a Shelby Tube in accordance with ASTM D-1587 (Standard Practice for Thin-Walled Tube Geotechnical Sampling of Soils)

- Grain size analyses of soils should be performed in accordance with ASTM D-422 (Standard Test Method for Particle-Size Analysis of Soils) for sieve and hydrometer analyses.

- Atterberg Limits tests should be performed in accordance with ASTM D-4318 (Standard Test Methods for Liquid Limit, Plastic Limit and Plasticity Index of Soils).

- Relatively undisturbed samples should be tested for in-situ moisture and density in accordance with ASTM D-2216 (Standard Test Method for Laboratory Determination of Water [Moisture] Content of Soil and Rock by Mass) and ASTM D-2937 (Standard Test Method for Density of Soil in Place by the Drive-Cylinder Method).

- Standard compaction tests for waste materials should be performed in accordance with ASTM D-698 (Standard Test Method for Laboratory Compaction of Soil Using Standard Effort; Standard Proctor).

- Relative density of cohesionless granular materials, if any, should be tested in accordance with ASTM D-4253 (Standard Test Method for Maximum Index Density and Unit Weight of Soils Using a Vibratory Table) and ASTM D-4254 (Standard Test Method for Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density).

- Direct shear tests of undisturbed and remolded soils should be performed in accordance with ASTM D-3080 (Standard Test Method for Direct Shear Test of Soils under Consolidated Drained Conditions).

- Slake-Durability tests, if performed on materials obtained from the waste rock piles, should be performed in accordance with ASTM D-5312 (Standard Test Method for Slake Durability of Shales and Similar Weak Rocks).

- Freeze-Thaw tests, if performed on materials obtained from the waste rock piles, should be performed in accordance with ASTM D-4644 (Standard Test Method for Evaluation of Durability of Rock for Erosion Control under Freeze-Thaw Conditions).

- Organic content of soils should be performed in accordance with ASTM D-2974 (Standard Test Methods for Moisture, Ash, and Organic Matter of Peat and Other Organic Soils).

- Monitoring wells will be installed in accordance with ASTM 5092 (Design and Installation of Ground Water Monitoring Wells in Aquifers).

- Vibrating wire piezometers will be installed in accordance with USBR or U.S. Army Corps of Engineers requirements.

- Borehole inclinometers, if any, will be installed and monitored in accordance with ASTM D-6230 (Test Method for Monitoring Ground Movement Using Probe-Type Inclinometers).

- Groundwater conditions in the Surface Mine and Waste Rock Pile Areas should be evaluated in accordance with ASTM D-5979 (Standard Guide for Conceptualization and Characterization of Ground-Water Systems).

5.6 Quality Control

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Quality control will be performed on a continuous basis by site personnel as work progress in the field. Field record books will be maintained as necessary and field logs will be maintained and copied daily to eliminate the possibility of lost data. Approximately 5 to 10 percent additional samples will be collected in the field, beyond those specified, for later testing if test results appear to be in error.

Samples will be handled, packaged, labeled and shipped to the testing laboratory in accordance with accepted ASTM and EPA standards. All testing by the laboratory will be performed in accordance with accepted ASTM standards including all required data and information reporting required by the standards.

Field logs of borings and test pits will be reviewed and corrected as necessary based on the laboratory testing. The geotechnical report will be developed by consultants for W.R. Grace and reviewed by the various parties involved in the program.

Surveying for location and elevation of borings and test pits will be performed in accordance with accepted survey standards of the American Congress on Surveying and Mapping (ACSM) and the National Society of Professional Surveyor (NSPS).

Table 5-1 – Summary of Geotechnical Investigations

Boring, Test Pit or Item ID	Bulk Samples	Undisturbed Samples	Index Tests	Moisture-Density Tests	Compaction Tests	Strength Tests	Rock Durability Tests	Organic Content Tests	Piezometer or Monitoring Well	Comm
TSF CPT-1	—	—	—	—	—	—	—	—	—	Std. C Rpt
TSF CPT-2	—	—	—	—	—	—	—	—	—	Std. C Rpt
TSF CPT-3	—	—	—	—	—	—	—	—	—	Std. C Rpt
TSF Existing Piezo. P-0	—	—	—	—	—	—	—	—	Install VW Piezo.	Repl Piezo
TSF-SM1	—	—	—	—	—	—	—	—	—	SM install on cre max secti
CTP-TP1	1-2	1	1-2	1	—	—	—	1	—	—
CTP-TP2	1-2	1	1-2	1	—	—	—	1	—	—
CTP-TP3	1-2	1	1-2	1	—	—	—	—	—	—
CTP-TP4	1-2	1	1-2	1	—	—	—	1	—	—
CTP Geologic Recon.	—	—	—	—	—	—	—	—	—	Possib SM/Inc
SMA-TP1	1-2	1-2	1-2	1	—	—	—	1	—	—
SMA-TP2	1-2	1-2	1-2	1			—	1	—	—
SMA-TP3	1-2	1-2	1-2	1	—	—	—	1	—	—
SMA-TP4	1-2	1-2	1-2	1	—	—	—	1	—	—
SMA-Existing MW-1 and MW-2	—	—	—	—	—	—	—	—	—	Moni GW Leve and Samp
WRP-B1	3-4	2-3	2-3	2-3	1	—	—	—	—	—
WRP-B2	2-3	2-3	2-3	2-3	1	1- DS	—	—	—	New M
WRP-B3	2-3	2-3	2-3	2-3	—	—	—	1	—	New M
WRP-TP1	1-2	1-2	1-2	1-2	—	—	1 F-T or S-D	1	—	—
WRP-TP2	1-2	1-2	1-2	1-2	1		1 S-D or F-T	1	—	—
WRP-TP3	1-2	1-2	1-2	1-2	—	—	1 F-T or S-D	1		—
WRP-TP4	1-2	1-2	1-2	1-2	1	1-DS	1 S-D or F-T	—	—	—
WRP-SM1-3	—	—	—	—	—	—	—	—	—	3 SM

Notes: TSF denotes Tailing Storage Facility
CPT denotes Cone Penetrometer Test
DS denotes Direct Shear Test.
SM denotes Settlement Monument
MW denotes Monitoring Well
F-T denotes Freeze-Thaw Test

CTP denotes Coarse Tailing Pile
SMA denotes Surface Mine Area
WRP denotes Waste Rock Pile

S-D denotes Slake-Durability Test
VW denotes Vibrating Wire Piezometer

REQUEST INFO FROM MWH ON TOPO EVALUATION
THICKNESS OF WRP : OTHER MINE
NEEDED TO LOCATE SETTLEMENT MONUMENTS

GEOPHYSICAL SURVEY
SEISMIC REFRACTION SURVEY OF WRP

ISSUE - WHETHER TO DO THIS SURVEY BEFORE
OR AFTER BOREHOLES ?

JOHN : NICK BOTH RECOMMEND

JOHN WILL GET INFORMATION TO NICK -

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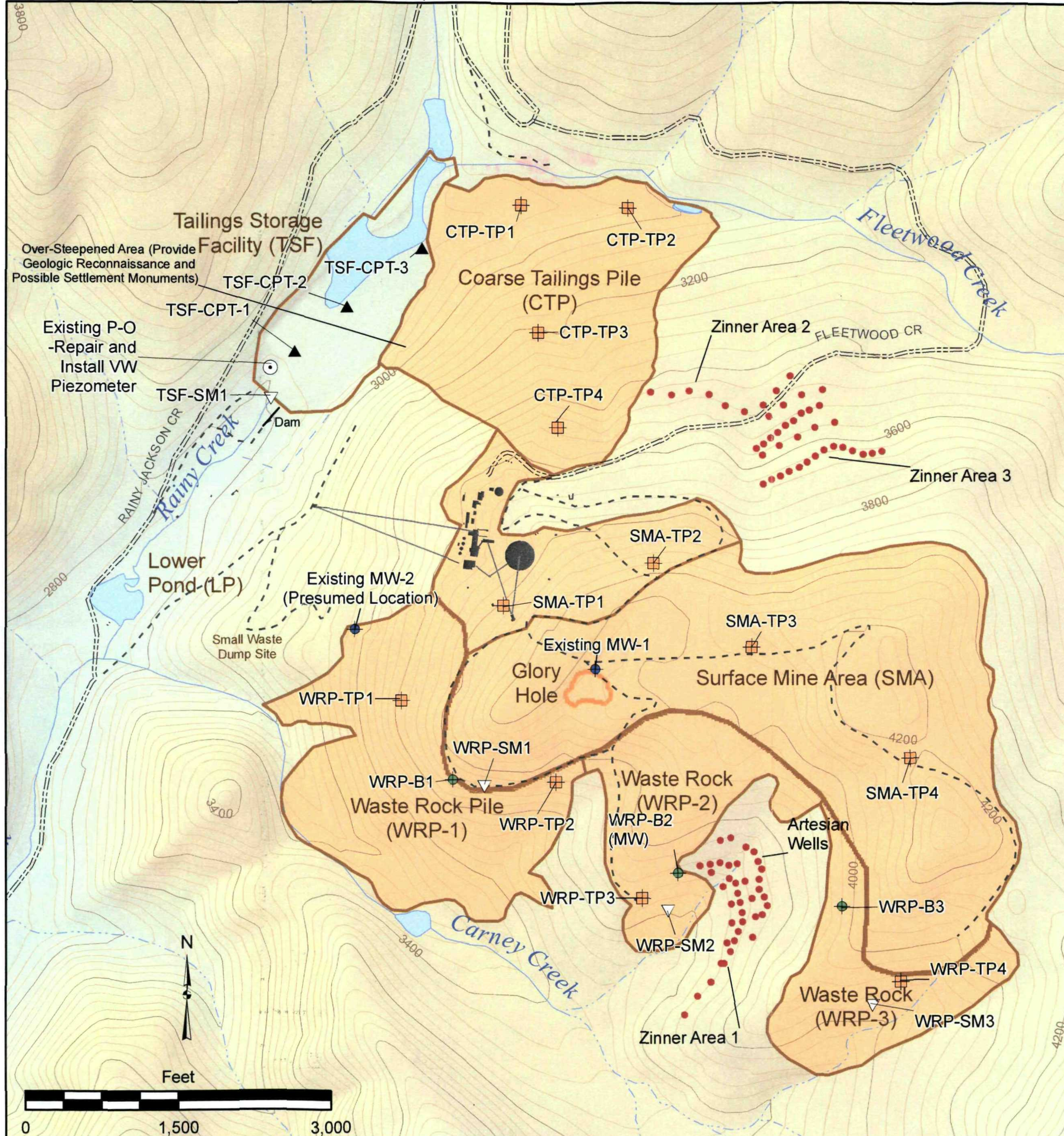

Legend

- County Road
- Primary Road
- MineRoads
- Glory Hole Area
- MineDistAreas
- Approximate Waste Rock Limit
- Mine Disturbance
- Zinner Boring Location
- Previous Monitoring Well
- Proposed New Borings (3)
- Proposed Cone Penetrometer Test (3)
- Proposed Test Pits (12)
- P-O Existing Non-Functional Piezometer Repair with New VW Piezometer
- SM-Proposed Settlement Monuments (4)

Note: Locations of borings and test pits are approximate and may vary in the field depending on access and field conditions encountered.

LIBBY MONTANA SUPERFUND SITE

OPERABLE UNIT 3

FIGURE 5-1

PHASE III SAP GEOTECHNICAL EXPLORATIONS FOR TAILINGS & MINE AREA

PROJECT: 0100-008-900	Apr 22, 2009	
REV: 0	BY: JHR	CHECKED: ACK

NEWFIELDS

ATTACHMENT 2

Section 5 – Other Data Needs

Input to Revisions for Data Quality Objectives, Section 5.3

Tailing Storage Facility

Geotechnical data have been developed previously for the Tailing Storage Facility dam for stability and safety evaluations. Such data appear to be acceptable for defining the general safety of the dam along with regular inspections and maintenance procedures. However questions remain regarding the facility and data is needed to answer such questions for FS evaluations, including:

- What is the thickness of tailings within the impoundment? This is required to estimate the volume of tailings. This question can be answered by performing Cone Penetrometer Tests (CPT) within the impoundment.
- What is the consistency of the tailings at various depths within the impoundment? This is required to evaluate stability and liquefaction potential and can also be answered by the CPTs.
- What are the piezometric conditions within the impoundment upstream from the dam embankment? This is required is determine the stability, seepage conditions and liquefaction potential of the impoundment. This can be answered by the CPTs in combination with repair and measurement of the existing piezometer (P-O) within the impoundment area along with installation of a vibrating wire piezometer at this location.
- What is a quantified amount of movement of the tailing dam over time? This is required to verify long-term stability in addition to visual assessments. This question can be partially answered by installation of a surface settlement monument on the dam crest.

[Would the diversion of Rainey Creek around the facility be a feasible, stable alternative? This question may be answered for the FS evaluations by a careful visual assessment of potential diversion dam locations upstream from the impoundment along with assessment of potential diversion channel routes followed by test pits if necessary <<*This would not be necessary for the No-Action Alternative but may be important to correctly assess the alternative which includes diversion of Rainey Creek.*]

Coarse Tailing Pile

Various questions remain regarding the Coarse Tailing Pile for FS evaluations including:

- What is the thickness of Coarse Tailing Pile materials? This is required to estimate the volume of materials within the Coarse Tailing Pile and can be answered with a series of test pits throughout the facility excavated to native materials beneath the tailing materials.
- What are the characteristics and variability of materials within and over the Coarse Tailing Pile? This is required to determine the long-term erosion potential and stability of the facility. This can be answered by sampling of materials and testing for index geotechnical characteristics such as grain size analysis and Atterberg Limits.
- What are the stability and conditions of the over-steepened area of the Coarse Tailing Pile? This is required to evaluate the long-term stability of the area. This may be answered by a complete initial geologic reconnaissance of the area based on standard protocol with an associated report, followed by surface settlement monuments or borehole inclinometers if determined to necessary.

Surface Mine Area

Various questions remain regarding the Surface Mine Area including:

- What is the wind and water erosion potential of the surface mine area? This is required for FS evaluations and may be answered by test pits and sampling near surface materials for index parameters.
- What is the global stability of the surface mine area. This is required to assess the long-term stability of the area particularly the area with benching and side slopes and can be answered by geotechnical analysis of samples from test pits.
- What is the volume of residential yard soils currently stored at the Surface Mine Area? This is required to determine the amount that may be used to place as cover over presently uncovered portions of the area. This can be answered by performing a ground survey of the soil stockpile.

Waste Rock Piles

Various questions remain regarding the Waste Rock Pile Area including:

- What is the thickness of the waste rock piles throughout the area? This is required to estimate the volume of the waste rock piles and can be answered utilizing data from boreholes developed through the wastes into the subsurface soils.

- What is the composition of the waste materials, particularly the amount of decomposing vermiculite? This is required to determine the overall strength of the waste piles which is important to know to assess the long-term stability of the piles. This can be answered by obtaining samples of the wastes from borings and test pits and testing the materials for index geotechnical parameters and rock/soil types.

- What is the in-situ density of fine-grained materials in the waste rock piles? This is important to know to assess the long-term stability and creep potential of the waste rock piles and their impact on the surrounding land. This can be answered by analyzing moisture and density of relatively undisturbed samples of materials obtained from boreholes and test pits and comparing them with compaction test data on disturbed samples of waste rock materials.

- What is the amount of LA asbestos in the waste rock piles at various depths? This is required to determine the potential release of materials into the environment and also the stability of the piles. This can be answered by sampling materials from various depths in borings and test pits and testing for LA.

- What is the impact of high groundwater and potential artesian conditions on the waste rock piles, particularly those adjacent to previously reported high groundwater and artesian conditions (between WRP-2 and WRP-3; Zinner "Area 1")? This is required to assess the impact of potentially high piezometric conditions on the long-term stability of the waste rock piles, and to assess the potential for hydraulic release of LA materials to the environment from high groundwater conditions. This can be answered by installing boreholes with monitoring wells into WRP-2 and WRP-3 adjacent to the previously reported high groundwater and artesian conditions between these waste rock piles.

- What is the quantified movement of the waste piles over time? This is important to know to verify existing stability of the piles in addition to visual assessments, and can be partially answered by installation of surface settlement monuments at key locations on the waste piles.

ATTACHMENT 3

5.0 OTHER DATA NEEDS FOR RI/FS

Additional geotechnical data are needed to support characterization of site conditions and evaluation of remedial alternatives in the FS. The long term effectiveness of the No Action alternative will require information to assess the stability of mine features and their potential to release materials into the environment. Potential source areas to be investigated are identified as follows:

- Tailing Storage Facility;
- Coarse Tailing Pile;
- Surface Mine Area; and
- Waste Rock Piles.

This section addresses the data requirements, data quality assessment, data quality objectives, sampling design, analytical requirements and quality control that are needed for the required geotechnical data at OU3.

5.1 Data Requirements

This section presents the background information necessary to assess the geotechnical engineering data requirements for OU3 RI/FS. This information is developed from various sources.

Tailing Storage Facility

The Tailing Storage Facility on Rainy Creek is impounded by a high-hazard, 135-feet high dam (127 ft reported by Harding, Lawson and Associates [HLA], 1992), initially constructed in 1971 with a 50-feet high starter dam (Schafer and Assoc., 1992). The dam is classified as high hazard due to its size and presence of hazardous constituents. The tailing dam is also known more recently as the Kootenai Development Impoundment Dam (Billmayer Engineering, Inc. 2007a) and previously as the W.R. Grace Vermiculite Tailings Impoundment or the W.R. Grace Dam, Rainy Creek, Montana (Schafer and Associates, 1992 and HLA, 1992). Most recently the tailing dam has been called the Kootenai Development Impoundment Dam (KDID) in the 2008-2009 periodic Owners' inspection report (Billmayer & Hafferman, 2009)

The Tailing Storage Facility covers an area of approximately 53 acres (75 acres at maximum flood pool), a portion of which contains open water area of several acres depending upon the inflow to the impoundment. The volume of impounded water at the emergency spillway crest is approximately 937 acre-feet and the volume at the dam crest is approximately 1,302 acre-feet (Schafer and Assoc., 1992). The impounded water is typically approximately 500 feet upstream of the tailings dam; however, during extreme flood events water could be impounded adjacent to the dam. The impounded water discharged over the spillway during the 2008 spring runoff period, and typically discharges during normal precipitation years.

The original tailing dam designer was HLA and Bovay Engineers, Inc. which performed the design in 1971 (Bovay Engineers, Inc. and Harding, Lawson Associates, 1971). Several drawings were reviewed from this design including design drawings from 1979 (W.R. Grace Co., 1979). The original design drawings indicate a 50-feet high starter dam with 2:1 side slopes, a 40-feet wide crest and a downstream chimney drain. The drainage system is shown starting upstream from the starter dam and extending along the foundation through each dam raise. Perforated cross drains are shown in the foundation for the Phase 1 and Phase 3 downstream embankment raises. Initial embankment materials are shown as "Zone 3 and abutment excavation" material, although no further description of these materials is given to identify if they are silty gravels, sands or clays etc. The drawings indicate three downstream raises of approximately 10 to 25 feet (Phases 1, 3 and 5) and two smaller centerline crest raises of approximately 5 to 10 feet (Phases 2 and 4). The fourth centerline raise to El. 2900 ft AMSL occurred in 1979 (Shafer and Associates, 1992). The fifth raise to El. 2926 ft AMSL is shown as a downstream raise apparently performed in 1981 (Billmayer & Hafferman, 2009). The downstream slope is shown as 2:1 with two benches each 10 feet wide. The centerline of the starter dam is shown as approximately 100-feet upstream from the 1979 dam crest centerline. The maximum design height of the embankment appears to have been 200 feet with downstream raises (Billmayer & Hafferman, 2009). Therefore, the current embankment height is approximately 67 percent of the final intended design height.

The crest length is approximately 1,100 feet, a concrete box culvert with principal spillway discharge is located on the left (east) abutment and an emergency spillway at a higher elevation is located on the right (west) abutment. The principal spillway has an outfall to Rainey Creek below the dam and the emergency spillway does not appear to have an outfall to the creek.

It appears from original drawings that foundation stripping up to about 5 feet in the valley bottom was performed to remove surface silts and that the abutments were stripped and benched. Original gravel blanket drains are shown in the design with perforated pipes to the downstream face, which were extended and added to during subsequent raises. Coarse tailing materials from the over-steepened area of the Coarse Tailing Pile were reportedly used in one or more of the dam raises. However, it is not clear where the coarse tailings might have been used in the embankment. Materials used for each embankment raise, whether centerline or downstream, are not defined in the available drawings.

A series of eight boreholes to maximum depths of about 55 feet below original ground surface and 4 test pits to a maximum of about 17 feet below ground surface (ft bgs) are shown on the 1971 drawings in the vicinity of the starter dam and downstream of the starter dam. The borings do not have SPT values and do not indicate consistency of materials (loose, dense, very dense etc.). Silt depths of up to 5 feet are indicated underlain by gravelly sand to sandy gravel of unknown consistency. The pyroxenite bedrock underneath the dam appears to be approximately 26-36 ft bgs. Bedrock on the right side near the west abutment appears to be deeper, about 40 to

45 ft bgs. Bedrock on the right abutment appears to be about 12 to 18 ft bgs and on the left abutment appears to vary from about 8 to 12 ft bgs. The rock is only about 1 ft bgs further up the left abutment area. A zone of silts and clays is indicated at depths of approximately 19 to 26 ft bgs and 35 to 37 ft bgs near the right abutment downstream toe. It is not indicated if these are soft zones. No test pits or borings are indicated in the impoundment area.

Tailings consist of interbedded layers of soft to stiff elastic silt (60%) and loose to medium dense poorly-graded sands and silty sand (40%) with mica and pyrite flakes. Based on two borings in the east side of the impoundment, the maximum thickness of tailings in the impoundment is approximately 70 to 75 feet (HLA, 1992). Confirmation of these depths and estimation of depth variations over the impoundment area, particularly further upstream, have not been performed. The loose silty sand tailing materials are reported to have liquefaction potential during seismic events (HLA, 1992).

Embankment soils reportedly consist of dense to very dense, well graded silty sands. The overall downstream embankment slope is shown on stability models to be approximately 4(horizontal):1(vertical), although existing slopes appear to be steeper than this. The right abutment is underlain by a thick blanket of glacial outwash and till from a few feet to 40 feet thick. The left abutment slope is blanketed by a relatively thin mantle of slope debris and remnants of a lateral moraine near the base of the canyon slope with an intermediate 4-feet thick zone of highly permeable, relatively clean sand. Natural foundation soils consist primarily of dense to very dense poorly-graded gravels, dense to very dense poorly-graded sands and moderately hard, friable pyroxenite bedrock with abundant magnetite and pyrite (HLA, 1992).

The surface discharge from the tailing impoundment discharges through a reinforced concrete principal spillway at a crest elevation of 2,897 ft AMSL located on the left (east) abutment. An inlet channel presently extends from the pond several hundred feet upstream from the dam crest to the principal spillway inlet. The principal spillway consists of an 8-feet wide by 4-feet high concrete box culvert approximately 169-feet long and an 8-feet wide by 3-feet high concrete discharge channel approximately 965-feet long with a concrete and riprap outfall. The principal spillway has a reported full-channel discharge capacity of 731 cubic feet per second (cfs) with the water surface at the dam crest. The concrete structures are reported to be partially cracked with some rocks and debris near the inlet.

The principal spillway and an emergency spillway, located on the right (east) abutment, are reportedly designed for one-half of the probable maximum flood (1/2-PMF; Schafer and Assoc., 1992). The peak inflow from the total Rainy Creek and Fleetwood Creek upstream drainage area (9.4 square miles) for the ½-PMF event was computed to be 5,838 cfs. The storage capacity of the impoundment was estimated to be approximately 1,302 acre-feet at the dam crest (Schafer and Assoc., 1992). Routing the ½-PMF flood hydrograph through the reservoir resulted in a peak discharge flow significantly lower than the peak inflow, and the present system of concrete principal spillway on one abutment and earth-riprap emergency spillway on the other abutment was recommended and constructed in the early 1990s. The emergency spillway is reported to be approximately 35-feet wide by 380-feet long with riprap erosion protection at an elevation of approximately 2,922 ft AMSL. The capacity of this emergency spillway with the water surface at the dam crest is reported to be 1,129 cfs (Billmayer & Hafferman, 2009). Thus, the combined discharge capacity of the principal and emergency spillways is approximately 1,860 cfs.

Recent risk-based analyses of the tailing dam concluded that the potential loss-of-life is 0.41 (Billmayer & Hafferman, 2009). Based on the current Montana spillway standards, the spillway design flow is therefore downgraded to an inflow design flood having a recurrence interval of 500 years. Analyses performed for this flood event determined the peak inflow from Rainey and Fleetwood Creeks to be 351 cfs utilizing USGS regression equations for Montana stream peak flows. This method provides an approximate method of determining peak flow with a standard error of prediction of approximately 67 to 79 percent. Therefore, based on this analysis, the existing peak design flow of 1,860 cfs for the spillway system, which is based on the ½-PMF inflow, is well in excess of the latest peak inflow from the 500-year flood event. Analyses were performed assuming loss of upstream vegetation due to a forest fire with a ground cover of approximately 20 percent. The peak inflows for this condition were estimated to be approximately 851 cfs. Environmental risk analyses have not been performed for the tailing storage facility.

By comparison, previous studies performed using a hydrograph analysis with full forest vegetation conditions and an overall hydrologic Curve Number of 60, estimated the combined peak flow from the 100-year, 24-hour storm event in Rainy and Fleetwood Creeks to be approximately 460 cfs (Schafer and Assoc., 1992).

Several open-tube piezometers are located in and near the dam embankment, which indicate either dry conditions or relatively low water levels. The maximum phreatic surface is reported to be approximately 94 feet below the crest of the dam, or approximately 40 feet above the base of the dam. One piezometer (P-2) is reported to fluctuate several feet each year and up to a maximum of approximately 33 feet. A piezometer at the dam toe (A-8) indicates piezometric surfaces varying approximately 5.5 feet at that location. The peak of the highest phreatic water surface each year corresponds to the peak of the snowmelt/rain runoff in the area in the late spring. Only one piezometer is located within the tailing impoundment, which is reported to have not been measurable the last few dam inspections. This piezometer (P-O) consists of a 2-inch diameter PVC casing with two ¼-inch tubes inside, and appears to require compressed air for reading (Billmayer & Hafferman, 2009).

A series of seepage-control pipes are located on the downstream embankment which have been maintained periodically (Billmayer, 2007a). The most recent dam inspections in December 2008 and January 2009 reported on each of the twelve seepage pipes exiting the downstream embankment (Billmayer & Hafferman, 2009). This report also discussed the various piezometers in and near the dam embankment. It was concluded that the drains and the phreatic surface indicated by the piezometers follow the yearly surface water flow fluctuations. It was further concluded that the majority of the volume of stream flow upstream of the tailing impoundment infiltrates the tailings and subsequently reports to the toe drainage system. A portion of the surface flow also discharges through the principal spillway during the late spring most years.

A surface water, piezometer and toe drain monitoring report was prepared for the water year 2008-2009 (Billmayer & Hafferman, 2010a). This report indicated approximately 1,154 acre-feet of water flowed into the tailing facility from Rainey and Fleetwood Creeks during the period from October 2, 2008 to October 23, 2009. Approximately 88% of this water discharged through the toe drainage system, 10% discharged through the principal spillway and 2% was lost to groundwater recharge. Therefore, the great majority of the inflow to the tailing facility reports to the toe drainage system. The maximum capacity of the toe drainage system is approximately 1,800 gallons per minute (gpm) and the base flow varies from approximately 200 to 400 gpm. This report concluded that: "*lacking a means to currently cut off drain flow or bypass inflow, the entire stability of the KDID will depend on the ability of the drains to discharge the water that infiltrates the reservoir and upstream face of the embankment. Therefore, the safety of the KDID will depend solely on making sure that there is always full drain flow capacity*" (Billmayer & Hafferman, 2010b).

The February 2010 B&H report described periodic water level data from 4 of the 12 piezometers located at the tailing dam from the dam crest to the toe. Some of the existing piezometers are not functional and some of the piezometers closer to the abutments remain dry. A summary of these piezometer data and other groundwater well data throughout the site are presented on Table 5-1. The dam piezometer data indicate that the majority of the seepage from the impoundment intercepts the main toe drain system along the centerline of the dam near the maximum section. The data also indicate that inflow to the impoundment has an almost immediate effect on the drain flow and a slightly delayed effect on the phreatic surface through the embankment. The recent highest phreatic surface through the embankment remained below the maximum phreatic surface modeled in the 1992 stability analysis.

Recent inspections of the interior of the twelve toe drains were performed using a video camera (Billmayer & Hafferman, 2010b). These inspections indicated that the toe drains are in fair to poor condition overall. Several of the drains were crushed or had gaps in the joints or other penetrations which allowed roots and moss in some of the drains and silt, sand and rock in other portions. One of the drains had turbid flow and another was transporting material out of the embankment. Inspections indicated that seepage flow was occurring outside some of the drain pipes and soft, wet areas are present near some drain outlets. Only the largest, central metal drain appeared to provide clear and unobstructed flow, although the pipe interior is corroded. This report recommended further investigations of the toe drain system.

B&H (2010a) recommended that "*the active piezometers and possibly drain flow could be monitored with transducers those transducers could be wired to a transmitter sending out real*

time data. Real time piezometer and transducer data would provide the best opportunity to have real time records and real time warnings when changes in these readings indicate problems." It was further recommended in this report that: *"a piezometer tube be installed adjacent to the spillway and that a transducer be placed in the piezometer and monitored for at least one if not two seasons."* This report also recommended that a flow measuring flume be installed in Fleetwood Creek and that additional inflow data to the tailing impoundment be obtained from Rainey and Fleetwood Creeks. Such data would likely provide important information for long-term planning of remedial alternatives at the tailing impoundment.
A groundwater monitoring well, Well C, is located downstream of the Tailing Storage Facility, approximately mid-way between the facility and the downstream Mill Pond. This is a 10-inch diameter well, which originally contained a pump, and is approximately 72-feet-deep. Groundwater levels in this well have varied from approximately 22 to 25 feet below ground surface.

Previous studies have concluded that the tailing embankment is stable during static and seismic conditions with acceptable deformations reported for an analysis assuming a maximum credible earthquake producing a horizontal ground acceleration of 0.30g (HLA, 1992). These analyses were based on the state of Montana standards prior to development of the new Montana Dam Safety Standards for High-Hazard Dams. Previous analyses appear to have utilized two-dimensional models in 1992. Recent stability analyses, with updated seismicity conditions, do not appear to have been performed for the structure. Finite element analyses of stress conditions utilizing state-of-the-art models, do not appear to have been performed for the dam and foundation.

Seepage through the dam has been identified as a potential long-term stability concern, particularly if the impounded water is adjacent to the dam. A levee was recommended in the 1992 HLA study to be located approximately 500 feet upstream from the dam crest to prevent the pond from reaching the dam; however, the levee was not constructed.

The Draft Environmental Assessment for the site (Montana Department of State Lands, 1992) identified a number of concerns with a full diversion of Rainy Creek around the Tailing Storage Facility including the following: *"The full diversion alternate increases the potential for failure, and decreases the safety of the system----Stability of the structure in a massive flood condition would be problematic---The channels carrying the diverted flows would be very large, and inherently less stable than smaller channels, particularly when constructed in the side of a hill as they would be in this case. From a hydrologic and geotechnical standpoint, any channel, natural or constructed, located above the low point in a drainage is generally not considered to provide good long-term service...Should diversion channels become plugged, or the system fail for some other reason, the flood flows would quickly breach the diversions and enter the impoundment".* This opinion was reiterated in the 1992 Schafer Engineering Analysis of Flood Routing Alternatives report.

It was noted in the February B&H report (2010a) that the tailing impoundment and dam structure, including the drainage system, are now decades old. It was further recommended in this report that: "*the feasibility of cutting off the drain flow, breaching the reservoir, by-passing the reservoir, or a combination of any or all or any other feasible means of decommissioning the KDID should continue to be investigated.*"

Geotechnical data including boring logs and laboratory testing were developed for this tailings dam during the 1992 study. Long-term maintenance of this tailings dam, to be evaluated in the FS, will be based on existing and additional data related to the geotechnical characteristics, confirmation of depth and extent of the tailings in the impoundment.

Coarse Tailing Pile

The Coarse Tailing Pile is located on the hillside east of the tailing impoundment and covers an area of approximately 140 acres. Based on topographic mapping, the total height of the Coarse Tailing Pile is approximately 700 feet and has side slopes of approximately 3:1 to 4:1. The Coarse Tailing Pile reportedly has had some reclamation procedures applied as discussed below. A small surface impoundment is located at the east toe of this pile covering an area of approximately 16,000 square feet (sf). Fleetwood Creek extends along the north toe of the Coarse Tailing Pile and storm flow events likely extend the floodplain over the toe of the Coarse Tailing Pile although specific hydrologic/hydraulic information was not identified for review.

A portion of the Coarse Tailing Pile appears to be at the slopes of 2:1 to 4:1 and a portion, approximately 65 acres, is reported to be too steep or over-steepened. This over-steepened portion was reportedly the borrow source for a tailing dam raise although documentation of this activity has not been identified. The northwest portion of the Coarse Tailing Pile extends into the upstream portion of the tailing impoundment and may have stability concerns.

Groundwater level data beneath the Coarse Tailing Pile and in the surrounding areas were not available for review. Groundwater level data for a number of borings are presented in the 1982 Zinner Report, which included a zone near the upgradient portion of the Coarse Tailing Pile area extending approximately 2,500 feet to the east (Figure 5-1). These indicated groundwater levels varying from approximately 41 to 146 feet below ground surface in the summer of 1981 (Table 5-1).

The Coarse Tailing Pile has reportedly undergone reclamation work including run-on control, contouring for runoff control, seeding, and planting of trees (Ray, 1999). The existing reclamation work has not been reviewed as part of this data needs assessment. This reclamation work was, however, reviewed for bond release by the Montana Department of Environmental Quality (MDEQ, 1999a).

A portion of the Coarse Tailing Pile reportedly experienced snowmelt/rain runoff erosion in 2007. This area reportedly required an estimated 6,500 cubic yards (cy) of restoration fill from nearby waste rock and relocation of an under-road culvert which apparently caused the washout (Remedium, 2007). Information regarding implementation of this erosion restoration was not identified for review.

Geotechnical data for the Coarse Tailing Pile were not identified, other than anecdotal descriptions. Various issues were raised following bond release in 1999 including comparable stability and utility of reclaimed areas and levels of asbestos on the surface of reclaimed areas and potential for continuing release (MDEQ, 1999b). It appears that insufficient data exist to adequately assess the long-term stability of the over-steepened area near the Coarse Tailing Pile. Such data will include geologic reconnaissance and settlement monuments in the over-steepened area followed by inclinometers if determined to be necessary based on the reconnaissance and settlement study. Geotechnical index parameters will need to be obtained from test pit samples in the coarse tailing area to determine engineering characteristics and the test pits will assist in determining the volume of the coarse tailing pile. Groundwater data beneath the Coarse Tailing Pile do not exist and piezometers will need to be installed in the north portion of the area to assess groundwater conditions.

Surface Mine Area

The Libby Mine site is located on top of a mountain that is part of the Rainy Creek Igneous Complex and is the upper portion of a hydrothermally altered igneous pyroxinite complex, which intruded into the Precambrian Belt Series rock. "Vermiculite Mountain" is generally a biotite/vermiculite deposit occurring in a pyroxenite matrix. Intrusions of syenite and pegmatite, which originated from a nearby synenite body, lie within the deposit. The vermiculite ore body at the Libby Mine contains various quantities of non-asbestos amphiboles as well as quantities of asbestiform or fibrous amphiboles.

The Surface Mine Area covers an area of approximately 270 acres at the top of the mountain. The disturbed area of the Surface Mine Area is contiguous with the mine waste rock piles immediately to the south. The former mill area was located just west of the Surface Mine Area and all associated facilities have been removed.

The Surface Mine Area includes the former "Glory Hole" which covers an area of approximately 15 acres southeast of the former mill area and adjacent to the Waste Rock Pile area. This was reportedly filled with miscellaneous mine waste debris (typical Class II landfill material), then covered and seeded as part of reclamation (Ray, 1999).

The Surface Mine Area was reportedly reclaimed in the 1990s including regrading, seeding and planting (Ray, 1999). This area was inspected for bond release in 1999 by the MDEQ. Issues remaining included levels of asbestos on reclaimed areas and water quality concerns related to potentially hazardous materials disposed in the Glory Hole and other areas (MDEQ, 1999b).

A stockpile of soil removed from residential yards in Libby is located on the Surface Mine Area as shown on Figure 5-2. The existing volume of this stockpile has not been identified.

Previous geologic and hydrogeologic investigations conducted in this area provide limited information regarding the local groundwater flow systems. The available groundwater monitoring wells, along with observed springs, are shown on Figure 5-1. Two wells were installed by WR Grace in 2000: one monitoring well was installed adjacent to the Glory Hole (MW-1; Well E in later reports) and the other monitoring well was installed near the west toe of the old waste dump (MW-2; W.R. Grace & Co., 2000; Well H in later reports). Another well, Well D, is located in an old pump house building that may have served as potable water source for the mine.

Well E (MW-1) near the Glory Hole is a 2-inch PVC well screened from 235 to 250 feet bgs. The log of the monitoring well indicates approximately 4 feet of rock fill over approximately 16 feet of vermiculite with weathered pyroxenite below this to a depth of approximately 82 feet below ground surface. Biotite pyroxenite bedrock was identified from a depth of 82 feet to the bottom of the borehole at a depth of approximately 250 feet below ground surface. Groundwater was found at a depth of approximately 242 feet below ground surface and produced approximately 1 to 2 gallons per minute. Recent water-level measurements of groundwater in Well E (MW-1) indicate groundwater levels varying from approximately 78 to 190 feet below ground surface. Groundwater level data are presented on Table 5-1.

Well H (MW-2) has a total depth of approximately 90 feet and is believed to be a more recent well located near a haul road on the hillside west of the mine. The well is also a 2-inch PVC, screened from 60 to 70 feet bgs. The well log indicates topsoil containing vermiculite for the first 5 feet overlying gravelly sand to approximately 15 feet bgs. Loose, fine sand with mafic minerals and areas of vermiculite are present to the bottom of the well. During groundwater sampling in July 2008 the total depth of the well was measured at 71 feet. Groundwater was identified in Well H (MW-2) during well construction at approximately 56 feet bgs and was reportedly high in arsenic and lead (MDEQ, 2000), although such data were not identified. Monitoring data from Well H in 2008 indicate groundwater levels approximately 60 feet to greater than 71 feet (dry conditions) below ground surface.

Well D is located at the bottom of a 5-foot-diameter culvert that extends approximately 8 feet below and 2 feet above the surrounding ground surface. The well is 10 inches in diameter, is cased with steel, and is screened from 345 to 385 feet bgs. The well initially was drilled to a total depth of 405 feet, then backfilled to 385 feet. Measurements in July 2008 indicate that the total depth extends to approximately 378 feet bgs with a soft sediment bottom. The well log indicates fill material to a depth of 37 feet bgs overlying vermiculite to 157 feet bgs. Pyroxinite and biotite pyroxinite bedrock were indicated from 157 to 215 feet bgs with a zone of dike material consisting mainly of quartz until approximately 378 feet bgs. The last 5 feet (to a total depth of 405 feet bgs) of the boring contained vermiculite. Data collected during construction indicated water levels at approximately 240 feet bgs and production of around 30 gpm. Measurements of groundwater levels in 2008 indicated levels approximately 241 to 244 feet bgs.

No records of groundwater monitoring wells or water level data have been identified in the eastern portion of the Surface Mine Area.

An earlier geo-hydrologic study was performed in areas north and south of the mine area that included drilling of approximately 100 boreholes to depths up to approximately 170 feet below ground surface (Zinner, 1982). These boreholes were located in the area between Waste Rock Piles 2 and 3 south of the mine and in an area east of the coarse tailing pile north of the mine. The boreholes indicated overburden materials from near zero to a maximum of approximately 90 feet bgs. Vermiculite pyroxenite was found below the overburden in thicknesses varying from approximately 40 to 170 feet bgs. Biotite pyroxenite bedrock was found from approximately 40 to 190 feet bgs.

Groundwater levels in these boreholes varied from the ground surface, with artesian conditions in the area between the waste rock piles, to approximately 140 feet below ground surface. The twelve artesian boreholes produced approximately 1 to 2 gpm water flow with release of trapped gas. Two boreholes north of the mine produced water flows of up to approximately 20 gpm. It was theorized that *"the aquifer is probably the result of a permeable zone of sandy and gravelly till overlain by a less pervious till"* (Zinner, 1982 [Harding and Lawson, 1974]). Zinner theorized that *"the artesian conditions are thereby the results of the upper inclination of glacial deposits to the canyon head where recharge takes place"* (Zinner, 1982). Confirmation of these theories has not been made and the areal extent of such conditions has not been determined.

As part of the same study, two deep boreholes were drilled in the mine area: one was drilled to a depth of 900 feet through the 22nd mining level (Hole 130) and one was drilled to a depth of 970 feet north of the mine area (Hole 131). The first deep borehole in the mine area indicated approximately 10 feet of overburden with 15 feet of vermiculite underlain by biotite pyroxenite to the 900 foot depth. This deep borehole produced approximately 25 gpm at the 500-foot depth, approximately 350 to 500 gpm was produced from a depth of 700 feet and drilling was stopped at 900 feet as approximately 1,000 to 2,000 gpm were being discharged to the surface. The final water level was approximately 66 feet below ground surface indicating the water level was under piezometric conditions. Another deep borehole was reportedly drilled 200 feet from Hole 130 which was reported to be under artesian conditions discharging approximately 5 gpm (Zinner, 1982). The second deep borehole north of the mine (Hole 130) did not encounter strong water producing zones as did Hole 130, although approximately 25 gpm was reported at a depth of approximately 500 feet below ground surface. The location and logs of deep boreholes 130 and 131 were not provided in the Zinner report.

Anecdotal information regarding an underground mine beneath the Libby Surface Mine Area has not been confirmed. Information regarding such underground workings was not identified during this investigation.

No additional geotechnical data were identified for review from the Surface Mine Area.

Mine Waste Rock Piles

The mine Waste Rock Piles are located south of the surface mine and cover a total area of approximately 230 acres. The toes of the Waste Rock Piles extend southwest to Carney Creek in some locations and the side slopes appear to be roughly at the angle of repose, and some contouring has reportedly been performed. The Waste Rock Piles consist of three major piles south and southeast of the former mill. For the purposes of this investigation, the larger Waste Rock Pile located to the south of the former mill site is designated WRP-1, the middle pile is designated WRP-2 and the southeast pile is designated WRP-3.

Topographic maps indicate that the largest WRP-1 has a total height in excess of 850 feet on the west side and has an overall slope of approximately 2:1 with haul roads and benches. Portions of the east side of WRP-1 and WPR-2 and WRP-2 have heights of approximately 150 to 200 feet at side slopes varying from 1.2: to 1.4:1. The existing hillside slopes vary from approximately 2.5:1 to 2.8:1. The topographic maps and aerial views (Google, 2010) indicate areas of gully erosion from the waste rock piles.

A small waste debris area, covering approximately 3 to 4 acres was located southwest of the mill. It was reported that miscellaneous debris (including drums) from this smaller Waste Rock Pile was disposed in an excavated area southwest of the mill site approximately 800 feet east of the Lower Pond (Ray Engineering, 1995). Water samples were reportedly obtained during reclamation of the small waste debris area but were not identified for review. This 1995 report also indicated movement of mine waste on the hillside thought to be caused by seepage from a spring and local areas of impounded water.

A land farm was reportedly developed for treatment of wastes from a leaking underground storage tank at or near the mill site. Information and data for this land farm treatment facility were not identified for review.

The Waste Rock Piles are reported to have undergone reclamation activities in the 1990s similar to the Coarse Tailing Pile and Surface Mine Area although the degree of reclamation is not known. A landslide area at one of the Waste Rock Piles covering approximately 45 acres exposed an old landfill in the 1990s, which was apparently reclaimed and the landfill debris was relocated elsewhere. The MDEQ reported that the landslide area had dried out and appeared to have stabilized (MDEQ, 1999a). However, hillside springs may re-appear at various locations depending upon snowpack and other factors. Decomposing vermiculite is typically a very weak material and its presence within the waste rock piles would tend to weaken the overall structures, particularly over time.

As discussed above, the Zinner report indicates artesian conditions in several of the boreholes between WRP-2 and WRP-3. It is not known how this artesian groundwater condition affects the stability of the waste rock piles. The Zinner report observed that the *"load created by the waste dumps and their impedance of water flow has created instability in the surrounding slopes and in the valley bottom"* (Zinner, 1982). Several springs have been located on the hillside between WRP-2 and WRP-3, which appear to confirm the presence of artesian conditions in this vicinity.

Well F was identified at the top of WRP-1, which is presently in poor condition. One groundwater data point was available for this well in October 2007 which indicated a groundwater level approximately 214 feet below ground surface. Well F is reportedly not in service at this time due to poor conditions of the well, and it is believed Well H, located approximately 2000 feet northwest of Well F, may be a suitable surrogate (Phase II SAP part B).

One groundwater monitoring well, Well A, is located just north of Carney Creek below WRP-1 and WRP-2. Well A indicates groundwater varying from less than one foot below ground surface during portions of the year to approximately 3 feet bgs.

A 1992 environmental assessment determined that *"the waste rock dump has inherent stability problems due to the structure of the ore and waste rock. The dump is currently standing at the angle of repose (1.25 to 1.5:1)....As a result of mass wasting, the waste rock dump toe has encroached on the Carney Creek stream channel. The slumping of waste rock has forced the creek to cut a new channel through the waste rock that has rolled to the bottom of the drainage in the end dumping process used to form the waste rock dump"* (Montana Department of State Lands, 1992).

Geotechnical data for the Waste Rock Piles were not identified and it appears there are insufficient data to assess the long-term stability of the facilities in the FS. Such data needed for analysis will include bulk samples for index parameters, compaction characteristics and strength parameters. Investigations will include test pits and geotechnical borings.

5.2 Data Quality Assessment

This data quality assessment includes a review of the identified engineering data for the Tailing Storage Facility, which primarily includes data for the impoundment dam related to stability and safety, and for the Surface Mine Area, which includes limited monitoring well data. Limited engineering data quality assessment is included for the Coarse Tailing Pile and the Waste Rock Pile areas based on very limited data adjacent to the areas.

Tailing Storage Facility

Because of the high-hazard rating of the tailing impoundment dam, geotechnical stability and hydrologic reports were completed for the facility in 1992 and periodic safety inspections have been performed since that time. Periodic safety inspections have found the structure to be safe with the implementation of additional maintenance measures associated with the downstream drainage system and with the addition of a reinforced concrete box culvert outlet through the left abutment and concrete discharge flume and chute downstream of the dam.

The geotechnical report completed in 1992 included 10 geotechnical borings to depths ranging from approximately 22.5 to 77 feet below ground surface (ft bgs). The soils were classified in accordance with American Society of Testing and Materials (ASTM) Standard D-2487 and visual-manual procedures were performed in accordance with ASTM D-2488. Standard Penetration Tests (SPTs) were performed in the borings in accordance with ASTM D-1586. Selected disturbed and undisturbed soil samples were tested for moisture content, dry density, Atterberg Limits, gradation, percent passing the No. 200 sieve, unconsolidated-undrained triaxial shear strength, consolidation and compaction characteristics. Although the testing procedures were not reviewed in detail, they were reportedly performed in accordance with established ASTM procedures.

The original design in 1971 included 8 geotechnical borings and 14 test pits in the vicinity of the starter dam and downstream of the proposed dam embankment. No explorations were performed upstream in the impoundment area. The borings determined the depth to bedrock and the test pits indicated near surface conditions. Standard penetration data were not reported for the borings and the general subsurface conditions were described from the boring and test pits logs presented on the design drawings. It is not known what quality control procedures were utilized the sampling and analysis of subsurface materials.

Twelve piezometers at the tailing dam have been monitored during the periodic safety inspections. All of these piezometers was monitored in the 2007, 2008-2009 and 2010 inspection reports (Billmayer & Hafferman, 2009 and 2010a). One additional piezometer not measured is apparently located in the impoundment area approximately 300 feet northeast of the dam crest. Annual monitoring of the piezometers have reportedly found the phreatic surface in the dam to be relatively low, with a maximum height of approximately 3 to 4 feet above the dam foundation (HLA, 1992 and Billmayer, 2007a). Seven of the thirteen piezometers monitored contained water during the 2007 annual inspection and the latest inspection reported similar conditions. Real-time piezometric data for the dam has not been performed because transducers and data loggers have not been utilized in the open-tube piezometers.

The 2007 inspection report concluded that the dam was in good to excellent condition and that no significant structural or maintenance concerns were found that would require immediate action (Billmayer, 2007a). The emergency action plan, operational plan, routine maintenance plan and piezometer monitoring logs were reported to be up-to-date and effectively addressed the structure and its components. The annual dam safety inspections have reportedly been approved by the Dam Safety Program of the Montana Department of Natural Resources (DNRC).

The 2007 dam inspection report recommended cleaning the seepage outlet drains and performing minor maintenance work on the dam and concrete box culvert and chute spillway, some of which was described in a Montana 310 permit application (Billmayer, 2007b). Some of this work has apparently been performed and recent photographs of the inside of some drain pipes indicate some corrosion and deterioration (Billmayer & Hafferman, 2009). Long-term effectiveness of the existing dam drainage system has not been performed and is suspect due to the corrosion of some drain pipes. The most recent toe drain inspection report (Billmayer, 2010b) indicates that the toe drainage system is in generally fair to poor condition and the report recommended further field investigations of this system.

The 2007 dam inspection report also recommended that a review of bank stability and seismic stability be performed (Billmayer, 2007a). Documentation of this review has not been identified. The 2007 inspection report also recommended that preparation for the 5-year operational permit renewal inspection be conducted no later than the fall of 2008. These recommendations included: 1) development of a complete catalog of all available documentation and reports for the tailing dam, 2) a complete review of the stability analysis based on the latest piezometer data, and 3) a review of the seismic stability of the embankment based on the new Montana Dam Safety Seismic standards for high-hazard dams in Montana.

Recent stability assessments have relied on previous geotechnical field investigations, laboratory analyses of materials and stability analysis models. The most recent inspection report (Billmayer & Hafferman, 2009) included a review of the 1992 seismic stability study by Harding Lawson. However, a critical review of updated seismic information was not apparently performed for the dam; the latest report stated agreement with the previous analyses performed in 1992.

Verification of foundation conditions at the dam have not been performed, the original borings and test pits at the dam site were performed in 1971 and the most recent geotechnical borings in the vicinity of the dam were performed in 1991. Bedrock cores were not obtained and rock quality designations (RQDs) were not performed for the dam foundation. Additional stability analyses using recent state-of-the-art two-dimensional models have not been performed for the structure nor have finite element analyses of the dam structure stress conditions been performed.

Data regarding embankment movement over time has not been identified. There do not appear to be any surveyed settlement monuments on the dam crest; only visual assessments of embankment movement and erosion have been performed.

Some discrepancies exist regarding previous hydrologic analyses performed for the Tailing Storage Facility (Schafer and Assoc., 1992) and recent hydrologic analyses of inflow design floods (Billmayer & Hafferman, 2009). The USGS regression equation methodology utilized in recent analyses likely does not have the accuracy required (67-79% std. error) for a structure such as the Tailing Storage Facility Dam at the Libby Mine.

Surface Mine Area

A few boreholes are reported to have been performed in the Surface Mine Area including some deep boreholes, although engineering-geologic data were not identified for the boreholes. One geologic log was identified for the monitoring well adjacent to the Glory Hole (MW-1; Well E) in the Surface Mine Area. It is not known what procedures were utilized in measurement of groundwater levels and what quality control procedures, if any, were utilized in the sampling and analysis of groundwater from the monitoring wells. Groundwater well sampling has been performed as part of the RI and various data gaps appear to exist for groundwater level data. The log of reported MW-2 (Well H) was not identified for review.

Insufficient geotechnical data exist in the Surface Mine Area to characterize site conditions with the objective of supporting evaluation of remedial alternatives in the FS.

Data and information regarding reported underground mine workings and how such workings may affect the surface mine area, or other site areas, have not been identified.

Coarse Tailing Pile

As mentioned above, no geotechnical engineering data were identified for the Coarse Tailing Pile other than anecdotal information regarding grain size of the coarse tailing materials. A geo-hydrologic report performed in the early 1980s (Zinner, 1982) presented general subsurface logs for areas east of the Coarse Tailing Pile, north of the surface mine. These indicated varying groundwater levels east of the Coarse Tailing Pile, but data was not identified to define groundwater levels within the Coarse Tailing Pile area. It is not known what quality control procedures were utilized in measurement of the groundwater levels or in characterization of subsurface materials.

The general quality and amount of data in the Coarse Tailing Pile Area, including surface and subsurface geotechnical and groundwater level data, are insufficient for analysis of FS alternatives.

Waste Rock Piles

Geotechnical data were not available for the waste rock piles and only one groundwater level data point was available at one of the waste rock piles. The Zinner report indicated artesian conditions between two waste rock piles (WRP-2 and WRP-3) but did not define the lateral extent of such conditions. Springs in the vicinity of the previous Zinner borings appear to confirm artesian conditions, although no additional monitoring well data is available in the vicinity. It is not known what quality control procedures were utilized in the collection of data prior to 2007.

The general quality and amount of data in the Waste Rock Piles Area, including surface and subsurface geotechnical and groundwater level data, are insufficient for analysis of FS alternatives.

Available groundwater level data for the site are presented in Table 5-1.

5.3 Data Quality Objectives

Data quality objectives (DQOs) define the type, quality, purpose and intended uses of data to be collected (EPA, 2006). The seven steps involved in the DQO process will be followed to provide an effective project plan and to provide sufficient information to support key decisions regarding remedial alternatives. The DQO process developed by EPA includes the following seven steps : 1) State the problem that the study is designed to address, 2) Identify the decisions to be made with the data obtained, 3) Identify the types of data inputs needed to make the decision, 4) Define the bounds (in space and time) of the study, 5) Define the decision rule which will be used to make decisions, 6) Define the acceptable limits on decision errors, and 7) Optimize the design using information identified in Steps 1-6.

Statement of Problem

Remedial alternatives (including No Action) to be identified and evaluated in the FS require a sufficient amount of engineering information to support the evaluation of implementability, effectiveness and cost. Various remaining questions need to be addressed for each of the areas, including the Tailing Storage Facility, the Coarse Tailing Pile, the Surface Mine Area and the Waste Rock Pile Area, to be evaluated in the FS.

Tailing Storage Facility:

Geotechnical data have been developed previously for the Tailing Storage Facility dam for stability and safety evaluations. Such data appear to be acceptable for defining the general safety of the dam along with regular inspections and maintenance procedures. However questions remain regarding the facility and additional data are needed to answer remaining questions for FS evaluations, including:

- What is the thickness of tailings within the impoundment? This is required to estimate the volume of tailings. This question can be answered by performing Cone Penetrometer Tests (CPT) within the impoundment.

- What is the consistency of the tailings at various depths within the impoundment? This is required to evaluate stability and liquefaction potential and can also be answered by the CPTs.

- What are the piezometric conditions within the impoundment upstream from the dam embankment? This is required is determine the stability, seepage conditions and liquefaction potential of the impoundment. This can be answered by the CPTs in combination with repair and measurement of the existing piezometer (P-O) within the impoundment area along with installation of a vibrating wire piezometer at this location.

- What is the current state of consistency (density, softness etc.) of dam embankment and foundation materials? This is required to update stability analyses and determine the current overall stability of the dam. This can be answered by a deep geotechnical boring through the maximum dam section into the underlying foundation bedrock along with associated sampling and geotechnical testing of various samples.

- What are the real-time piezometric variations in the dam embankment? This is required to better determine the potential rapid drawdown conditions within the embankment for stability analyses and the effects of varying piezometric conditions on the tailing impoundment and embankment. This can be answered by installing pressure transducers in the new boring and in at least two existing piezometers within the embankment with data loggers and possibly remote data transmittal.

- What are the verified foundation conditions for the tailing dam including the consistency of materials through the maximum dam section and the bedrock conditions beneath the dam? This is required for updated stability analyses of the tailing dam. This question can be answered by a deep borehole through the dam maximum section with sampling and geotechnical testing.

- What is a quantified amount of movement of the tailing dam over time? This is required to verify long-term stability in addition to visual assessments. This question can be partially answered by installation of a surface settlement monument on the dam crest.

Coarse Tailing Pile:

Various questions regarding the Coarse Tailing Pile remain for FS evaluations including:

- What is the thickness of Coarse Tailing Pile materials? This is required to estimate the volume of materials within the Coarse Tailing Pile and can be answered with a series of test pits throughout the facility excavated to native materials beneath the tailing materials.

- What are the characteristics and variability of materials within and over the Coarse Tailing Pile? This is required to determine the long-term erosion potential and stability of the facility. This can be answered by sampling of materials and testing for index geotechnical characteristics such as grain size analysis and Atterberg Limits.

- What is the groundwater level within or beneath the Coarse Tailing Pile? This is required to determine the piezometric conditions for assessment of long-term stability of the facility. This can be answered with installation of piezometers in two of the test pits with screened interval spanning the base of the tailing and original ground surface.

- What are the stability and conditions of the over-steepened area of the Coarse Tailing Pile? This is required to evaluate the long-term stability of the area. This may be answered by a complete initial geologic reconnaissance of the area based on standard protocol with an associated report, followed by surface settlement monuments or borehole inclinometers if determined to necessary.

Surface Mine Area:

Various questions remain regarding the Surface Mine Area including:

- What is the global stability of the Surface Mine Area? This is required to assess the long-term stability of the area, particularly the area with benching and side slopes, and can be answered by test pits with limited geotechnical testing of samples and by performing visual assessments of the benches and existing conditions in the steep portions of the area.

- What are the groundwater levels in the north and east portions of the Surface Mine Area? This is required to provide a better understanding of the overall potentiometric conditions throughout the mine area, and can be partially answered by restoration of Well J in the north part of the area.

- What is the volume of residential yard soils currently stored at the Surface Mine Area? This is required to determine the amount that may be used to place as cover over presently uncovered portions of the area. This can be answered by performing a review of the amount of soils hauled to the site or possibly a ground survey of the soil stockpile if necessary.

Questions regarding the reported underground mine workings may need to be addressed during the FS. However, no field explorations associated with this are recommended at this time.

Waste Rock Piles:

Various questions remain regarding the Waste Rock Pile Area including:

- What is the thickness of the waste rock piles? This is required to estimate the volume of the waste rock piles and can be answered utilizing data from boreholes developed through the wastes into the subsurface soils and the possible use of seismic refraction surveys on the waste rock piles.

- What is the composition of the waste materials, particularly the amount of decomposing vermiculite? This is required to determine the overall strength of the waste piles which is important to know to assess the long-term stability of the piles. This can be answered by obtaining samples of the wastes from borings and test pits and testing the materials for index geotechnical parameters and rock/soil types.

- What is the in-situ density of fine-grained materials in the waste rock piles? This is important to know to assess the long-term stability and creep potential of the waste rock piles and their impact on the surrounding land. This can be answered by analyzing moisture and density of relatively undisturbed samples of materials obtained from boreholes and test pits and comparing them with compaction test data on disturbed samples of waste rock materials.

- What is the amount of LA asbestos in the waste rock piles at various depths? This is required to determine the potential release of materials into the environment and also the stability of the piles. This can be answered by sampling materials from various depths in borings and test pits and testing for LA.

- What is the impact of high groundwater and potential artesian conditions on the waste rock piles, particularly those adjacent to previously reported high groundwater and artesian conditions (between WRP-2 and WRP-3; Zinner "Area 1")? This is required to assess the impact of potentially high piezometric conditions on the long-term stability of the waste rock piles, and to assess the potential for hydraulic release of LA materials to the environment from high groundwater conditions. This can be answered by installing boreholes with monitoring wells into WRP-2 and WRP-3 adjacent to the previously reported high groundwater and artesian conditions between these waste rock piles.

- What is the quantified movement of the waste piles over time? This is important to know to verify existing stability of the piles in addition to visual assessments, and can be partially answered by installation of surface settlement monuments at key locations on the waste piles and with borehole inclinometers.

Identify the Decision

The engineering data collected during the OU3 RI Phase III is intended to help EPA decide if and what remedial alternatives are feasible and necessary to protect human health and/or ecological receptors from unacceptable risks from asbestos and any other mining-related contaminants at the Tailing Storage Facility, Surface Mine Area, Coarse Tailing Pile, and Waste Rock Piles over the long term.

Identify Types of Data Needed

Engineering data needed for the various areas at OU3 include:

- Boring logs and test pits with associated logging in accordance with generally accepted ASTM standards and Cone Penetrometer Testing (CPT) in the tailing impoundment area;

- Subsurface soil sampling for bulk samples and relatively undisturbed samples;

- Geotechnical laboratory testing for index parameters such as grain size analysis and Atterberg Limits and strength/durability characteristics as necessary depending upon location of sampling;

- Installation of piezometers and groundwater monitoring wells for routine measurement and assessment of groundwater and phreatic surfaces through the various facilities;

- Geologic reconnaissance and field inspection of existing conditions is needed in some areas as a first step in evaluation of long-term stability;

- Installation of settlement monuments, or borehole inclinometers if determined to be necessary, at various locations to assess long-term embankment and waste pile/hillside stability concerns; and

- Survey data to determine the location and elevation of borings, test pits, monitoring wells, piezometers and settlement monuments or inclinometers and to verify existing slope conditions at the facilities.

- Seismic refraction survey information to define general subsurface conditions at the site.

Define Bounds of Study

The spatial bounds of the study include the total areas currently occupied by the Tailing Storage Facility, Coarse Tailing Pile, Surface Mine Area, and Waste Rock Piles at OU3.

The temporal bounds of the study will include one season of geotechnical sampling and monitoring new monitoring wells and settlement monuments, or borehole inclinometers, as applicable during a typical range of annual groundwater conditions.

Define the Decision Rule

The quality and results of engineering data from OU3 will not be used to determine if remedial action is necessary. However, used in combination with the decision rules for human and ecological risks and for potential environmental impacts, the data will be used to support FS evaluations.

Define Acceptable Limits on Decision Errors

Acceptable limits on decision errors for engineering data from OU3 will be based on established engineering principals, accepted ASTM standards and engineering judgment. Typically, if data are within reasonable limits for the type of material sampled and within the range of previous data for similar materials or previous data for the facilities, the data will be accepted.

Optimize the Design

The sampling design is based on the DQO process, the site characteristics and scale, and anticipated needs to support identification and evaluation of remedial alternatives in the FS process. Locations of investigation and sampling points may be varied somewhat in the field from the plan depending upon field conditions encountered.

5.4 Sampling Design

The sampling design includes various field geotechnical cone penetrometer tests, borings and test pits with associated logging, sampling and testing of soils, tailings and waste rock from the borings and test pits. The approximate location of the test pits and borings are shown on Figure 5-2 and the program is summarized in Table 5-2. Ranges of sample numbers are provided. The lower number indicates the minimum requirement. If the material is heterogeneous more samples than the minimum may be required based on field observation. Depending upon initial field investigations in various areas, additional geotechnical investigations may be necessary in addition to those indicated on Table 5-2. Such areas may include the potential diversion locations for Rainey Creek around the Tailing Storage Facility and the over-steepened area of the Coarse Tailing Pile.

Additional field geologic reconnaissance and inspection of existing conditions of various areas will also be performed as will land surveying of various features.

The location and elevation of all borings and test pits will be determined using survey-grade global positioning system (GPS) equipment. This equipment should provide the state plane coordinates to the nearest tenth of a foot and should provide the elevations to the nearest tenth of a foot based on feet above mean sea level.

All excavated test pits and boring cores will be documented with digital photography as necessary for each of the sampling locations.

Tailing Storage Facility

Previous investigations at the Tailing Storage Facility included a total of 10 borings developed for the 1992 geotechnical stability investigation of the impoundment dam. A total of 12 piezometers are annually monitored for dam safety inspections, none of which are within impounded tailings upstream of the dam. The original tailing dam design also included a series of borings in the vicinity of the dam which identify bedrock.

A total of three cone penetrometer tests (CPT) are proposed at the Tailing Storage Facility impoundment to verify the thickness and characteristics of the impounded tailing materials and subsurface conditions: at the upstream area (approximately 500 feet upstream of the embankment) where a levee was proposed in the 1992 report, one approximately 1,000 feet upstream from the dam and one approximately 2,000 feet upstream from the dam as shown on Figure 5-2. The location of these CPTs is approximate and may vary in the field depending upon accessibility.

Use of CPT methods should utilize low-ground-pressure equipment to access areas not possible with a conventional drill rig. This method does not extract samples of subsurface materials for laboratory testing, but rather utilizes electronic friction cone or piezocone equipment to record the penetration resistance of subsurface strata. This data presents a qualitative correlation to physical properties of materials present such as shear strength, bearing capacity, void ratios and pore pressures. Since data is continuously recorded, the depth, thickness and variation in the stratigraphy provide a complete profile of the materials encountered. The CPT data will be presented in standard format for each location with associated analyses of the data.

One deep geotechnical borehole should be drilled through the maximum tailing dam section at least 25 feet into underlying bedrock. This should be performed by a combination of auger rig and air rotary methods, as necessary, with sampling of embankment, tailing and bedrock materials. It is estimated that the depth of this borehole will be approximately 175 to 180 feet. Disturbed samples of drill cutting will be collected as will split spoon samples in liners and undisturbed thin wall tube samples. Two thin-wall (Shelby) tube samples will be collected of fine-grained tailing and embankment materials. One or two rock core samples will be collected of the bedrock beneath the tailing dam. Index tests (grain size analyses and Atterberg Limits) will be performed on several samples and in-situ moisture-density tests will also be performed on tube samples. One Standard Proctor Compaction test will be performed on a bulk sample of embankment material for comparison with in-situ moisture-density tests. One undisturbed tube sample will be tested for triaxial shear. The borehole will be converted to a piezometer with screened interval in the lower portion of the embankment just above the foundation. A transducer will be installed in the piezometer, along with a data logger, to record real-time piezometric conditions within the dam embankment.

The existing non-functional piezometer in the impoundment area (P-0) should be repaired as necessary to assess the piezometric conditions in that area. It is recommended that a vibrating wire piezometer be installed to monitor pore pressure changes in the tailing materials. Such instruments provide a better assessment of piezometric conditions than open-tube piezometers in fine-grained materials such as tailings. Vibrating wire piezometers will be stainless steel units with durable pressure transducers capable of measuring pore pressures from -50 to 1,000 kilopascals (kPa; 145 pounds per square inch, psi) with an accuracy of plus or minus 0.1% full range. The unit shall be hermetically-sealed with durable cables and data loggers as necessary. The piezometer will be adequately protected with locking steel casings and concrete collars as necessary.

At least two of the existing piezometers in the dam embankment should be modified with installation of pressure transducers to measure real-time piezometric changes in the embankment. These should be installed in P-2 and PM-2 at a minimum, with possible installation in A-8. Data loggers should be installed to record all data with possible remote readout capability.

At least one concrete settlement monument will be placed on the tailing dam crest at the maximum section and will be surveyed to establish baseline data. This will provide needed quantification of embankment movements to complement and verify visual assessments and piezometer readings during periodic inspections. This will be a 10-inch diameter by 48-inch deep concrete cylinder installed vertically with the top approximately 3 inches above the existing ground surface. It may be either cast-in-place or precast concrete constructed with concrete having a minimum 28-day compressive strength of at least 3,500 pounds per square inch (psi). The top surface will have an embedded brass survey marker and will be surveyed for horizontal and vertical control from existing benchmarks; to the nearest 0.01 ft. Subsequent surveyed readings should then be performed twice per year through the FS period and following final remedial action. A survey point on the existing concrete principal spillway structure should also be established with associated baseline data.

Visual assessments of existing ground conditions along potential diversion dam and diversion channel alignments for Rainey Creek upstream and adjacent to the Tailing Storage Facility will need to be made as a first step. If determined to be necessary during visual assessments, various test pits may be excavated at the potential diversion dam and channel locations with associated logging and sampling for index parameters.

Geophysical survey procedures may be performed at the Tailing Storage Facility as necessary if access by CPT equipment is not possible in portions of the impoundment. This would be a ground seismic refraction survey.

Coarse Tailing Pile

Geotechnical investigations for the Coarse Tailing Pile will require four test pits. Approximate locations of the test pits are shown on Figure 5-2. The location of the test pits may vary in the field depending upon accessibility.

The test pits will be excavated with a large backhoe (track-hoe) to depths of approximately 10 to 12 feet. Slopes of test pits will be laid back to provide safe conditions as required by OSHA. The test pits will be logged by an experienced geologist or geotechnical engineer. Bulk samples of coarse tailing materials and underlying materials will be obtained and relatively undisturbed hand-driven samples will be obtained as possible. The hand-driven samples will be collected in 2-inch diameter by 4-inch long brass or stainless steel tubes. Alternatively 3-inch diameter by 6-inch long brass or stainless steel tubes could also be used.

Two test pits should be excavated near the toe of the Coarse Tailing Pile: one approximately 100 to 200 feet west of the pond and another approximately 800 to 1,000 feet west of this. These should be excavated to the base of the coarse tailing. Another test pit should be excavated about mid-way up the Coarse Tailing Pile slope in a relatively stable area and another should be excavated near the top of the Coarse Tailing Pile.

Bulk samples of cover soils, coarse tailing and subsurface materials should be collected from three of the test pits, as applicable. These samples should be tested for index properties including grain size analyses and Atterberg Limits as necessary depending amount of fines in the sample. In general, if the sample contains less than 10 percent fines (silt and clay passing the No. 200 sieve), Atterberg Limits will not be required, and the grain size analyses only need to be on the plus 200 sieve sizes. A few index property tests will be performed on bulk samples and in-situ moisture-density tests will be performed on relatively undisturbed tube samples. In addition, a few samples of existing cover soils should be tested for organic content. An assessment of the areal extent and thickness of existing cover soils will be made for the entire Coarse Tailing Pile Area.

Two of the test pits in the Coarse Tailing Pile area, CTP-TP1 and CTP-TP2, will have open-tube piezometers installed to determine groundwater levels and fluctuations beneath the area. These test pits will need to be excavated deep enough to reach groundwater level beneath the coarse tailing. The piezometers will consist of a 1.5-inch diameter PVC tube installed vertically with a screened interval from the base of the tailing to the groundwater level likely below the original ground surface. Granular fill filter material will be placed around the screened interval and compacted backfill and tailing placed around the remainder of the piezometer.

A geologic reconnaissance will be performed in the over-steepened area of the Coarse Tailing Pile as a first step. This reconnaissance should evaluate all surface conditions including visible surface features, seeps, if any, and evidence of movement with associated digital photographic documentation. A land survey should be performed of the over-steepened area including the adjoining land on both sides, above and below the area. If determined to be necessary following the initial investigations, settlement monuments will be installed at selected locations to monitor movement of the area over time. If movement of the over-steepened area is occurring, inclinometer(s) may be installed to further evaluate movements at depth.

Surface Mine Area

The Surface Mine Area will be investigated with test pits as shown on Figure 5-2 and with visual assessments of the area. Two test pits are recommended in the Surface Mine Area with associated logging and sampling of cover soils, mine wastes and subsurface materials. The thickness of cover soils should be recorded at each location and the soil horizon should be logged as necessary.

Bulk samples of subsurface materials should be obtained for index testing from at least one test pit: grain size analyses and Atterberg Limits Additionally, a sample of cover soil should be tested for organic content. An assessment of the areal extent and thickness of existing cover soils will be made in the Surface Mine Area.

A review of the amount of residential soils currently stockpiled on the Surface Mine Area should be made. The existing stockpile of residential soils may be surveyed if necessary to obtain an accurate volume of such materials.

Existing groundwater monitoring wells at the Site are being sampled as part of the RI. Data from this sampling will be used in the assessment of conditions in the Surface Mine Area and Waste Rock Pile Area. Existing Well J should be restored to obtain groundwater levels in that area, if possible.

Data from existing monitoring wells Well E (MW-1), Well H (MW-2), Well D, Wells F and J (if possible), and previous well information from the Zinner Report, in addition to new monitoring wells to be installed will be utilized to gain a better understanding of the geo-hydrologic conditions in the Surface Mine-Waste Rock Pile-Carney Creek area. Conceptualization and characterization of the groundwater system in the project area will be performed in accordance with accepted standards.

Mine Waste Rock Piles

The three Waste Rock Piles will be investigated through a series of four test pits and three borings with two monitoring wells. The four test pits will include two on WRP-1 and one each on WRP-2 and WRP-3. Two or three of the test pits will be excavated near the top of the Waste Rock Piles and the remainder will be excavated in lower, accessible portions of the Waste Rock Piles.

One boring is proposed at the top of WRP-1 to assess the thickness of mine waste and subsurface soil horizon for stability. These borings should extend at least 5 feet into the native materials beneath the Waste Rock Pile for confirmation purposes. One boring each will be advanced through WRP-2 and WRP-3 within a few hundred feet of the previous borings which indicated artesian groundwater conditions. These should be located up-gradient and down-gradient of the previous boreholes performed in the Zinner Study Area 1. The exact locations will be field selected based on accessibility. Approximate locations of borings, monitoring wells and test pits shown on Figure 5-2 may vary in the field depending upon accessibility.

Two of the borings, in the WRP-2 and WRP-3 areas, will be developed as monitoring wells with 5 to 10 feet screened intervals within the groundwater zones encountered. It is anticipated that this will require 2-inch diameter Schedule 80 PVC casing. The MWs should be developed as necessary and monitored at least quarterly during the FS evaluation period. These monitoring wells should have protected steel pipe sections above ground surface with locking tops and concrete slabs at ground surface.

If possible, Well F should be rehabilitated to provide additional groundwater data between the surface mine area and the largest waste rock pile (WRP-1).

Three settlement monuments will be installed in the WRP areas to assess movement of these structures over time. These will be 10-inch diameter by 48-inch deep concrete cylinders installed vertically with the top approximately 3 inches above the existing ground surface. These may be either cast-in-place or precast concrete constructed with concrete having a minimum 28-day compressive strength of at least 3,500 pounds per square inch (psi). They will have brass survey markers embedded in the top and will be surveyed for horizontal and vertical control from existing benchmarks, to the nearest 0.01 ft.

One borehole inclinometer will be installed in WRP-B1. This inclinometer will allow an assessment of the overall movement of waste rock pile with depth. The inclinometer, along with the surface settlement monument located approximately 800 feet to the northwest, will provide an overall assessment of the movement of the largest waste rock pile over time.

Bulk samples of cover soils, waste rock and subsurface materials, as applicable should be obtained and tested for index parameters of grain size and Atterberg Limits, compaction and organic content of cover soils as necessary. Index tests will be performed on bulk samples, and a few organic content tests will be performed on surface soils and compaction tests will be performed on bulk, composite samples. The size of bulk samples may vary from large zip-lock plastic bags for index and organic content tests to 5-gallon bucket samples for compaction tests. An assessment will be made of the approximate volume of vermiculite in the Waste Piles based on visual assessments and sampling of borings and test pits.

Relatively undisturbed samples from borings or test pits will also be tested for in-situ moisture density. These in-situ moisture density tests will provide a definition of existing material conditions throughout the waste rock piles and some will be compared to the compaction tests to estimate the existing degree of compaction of materials. In addition, samples will be tested for strength to assess short and long-term stability of the Waste Rock Piles. The decomposition potential of materials within the waste rock piles will be evaluated through the use of freeze-thaw or slake-durability tests of selected samples of materials.

Geophysical survey methods may be utilized to determine subsurface conditions in areas between boreholes and in areas without any subsurface data. Such methods may consist of surface seismic refraction surveys or down-hole seismic surveys as applicable to the conditions.

Previous well information from the Zinner Report, in addition to new monitoring wells to be installed in boreholes (WRP-B2 and WRP-B3) will be utilized to gain a better understanding of the geo-hydrologic conditions in the Waste Rock Pile-Carney Creek area. Conceptualization and characterization of the groundwater system in the project area will be performed in accordance with accepted standards.

5.5 Analytical Requirements

The latest revision of the ASTM standards should be followed for all geotechnical soil and rock sampling and testing procedures. The following ASTM standards will be followed in sampling and analysis of geotechnical samples from OU3:

- Geotechnical Field Work should be performed in accordance with ASTM D-420 (Site Characterization for Engineering Design and Construction Purposes).

- Geologic reconnaissance procedures should be performed in accordance with standard ASTM procedures (Part 4.5 of ASTM D420-2003).

- Subsurface soils encountered in test pits and borings should be logged by an experienced geologist or geotechnical engineer in accordance with ASTM D-2487 (Classification of Soils for Engineering Purposes; Unified Soil Classification System) based on visual-manual procedures specified in ASTM D-2488 (Description and Identification of Soils; Visual-Manual Procedure).

- Standard penetration tests during boring shall be performed in accordance with ASTM D-1586 (Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils).

- Air rotary drilling will be performed in accordance with ASTM D-5782 (Standard Guide for Use of Direct Air-Rotary Drilling for Geoenvironmental Exploration and the Installation of Subsurface Water-Quality Monitoring Devices).

- Rock core drilling and sampling of rock beneath the tailing dam will be performed in accordance with ASTM D-2113 *(Standard Practice for Rock Core Drilling and Sampling of Rock for Site Investigations).*

- Downhole seismic testing will be performed in accordance with ASTM D-7400 (Standard Test Method for Downhole Seismic Testing).

- Seismic refraction investigations will be performed in accordance with ASTM D-5777 (Standard Guide for Using Seismic Refraction Method for Subsurface Investigations).

- Selection of geophysical subsurface investigation methods will be performed in accordance with ASTM D-6429 (Standard Guide for Selecting Surface Geophysical Methods).

- Cone penetrometer testing shall be performed in accordance with ASTM D-5778 (Standard Test Method for Performing Friction Cone and Piezocone Penetration Testing of Soils).

- Relatively undisturbed cohesive soil and tailings samples should be obtained using a Shelby Tube in accordance with ASTM D-1587 (Standard Practice for Thin-Walled Tube Geotechnical Sampling of Soils)

- Grain size analyses of soils should be performed in accordance with ASTM D-422 (Standard Test Method for Particle-Size Analysis of Soils) for sieve and hydrometer analyses.

- Atterberg Limits tests should be performed in accordance with ASTM D-4318 (Standard Test Methods for Liquid Limit, Plastic Limit and Plasticity Index of Soils).

- Rock core samples from beneath the tailing dam will be evaluated in accordance with ASTM D-5878 *(Standard Guide for Using Rock-Mass Classification Systems for Engineering Purposes).*

- Relatively undisturbed samples should be tested for in-situ moisture and density in accordance with ASTM D-2216 (Standard Test Method for Laboratory Determination of Water [Moisture] Content of Soil and Rock by Mass) and ASTM D-2937 (Standard Test Method for Density of Soil in Place by the Drive-Cylinder Method).

- Standard compaction tests for waste materials should be performed in accordance with ASTM D-698 (Standard Test Method for Laboratory Compaction of Soil Using Standard Effort; Standard Proctor).

- Relative density of cohesionless granular materials, if any, should be tested in accordance with ASTM D-4253 (Standard Test Method for Maximum Index Density and Unit Weight of Soils Using a Vibratory Table) and ASTM D-4254 (Standard Test Method for Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density).

- Direct shear tests of undisturbed and remolded soils should be performed in accordance with ASTM D-3080 (Standard Test Method for Direct Shear Test of Soils under Consolidated Drained Conditions).

- Slake-Durability tests, if performed on materials obtained from the waste rock piles, should be performed in accordance with ASTM D-5312 (Standard Test Method for Slake Durability of Shales and Similar Weak Rocks).

- Freeze-Thaw tests, if performed on materials obtained from the waste rock piles, should be performed in accordance with ASTM D-4644 (Standard Test Method for Evaluation of Durability of Rock for Erosion Control under Freeze-Thaw Conditions).

- Organic content of soils should be performed in accordance with ASTM D-2974 (Standard Test Methods for Moisture, Ash, and Organic Matter of Peat and Other Organic Soils).

- Monitoring wells will be installed in accordance with ASTM 5092 (Design and Installation of Ground Water Monitoring Wells in Aquifers).

- Vibrating wire piezometers will be installed in accordance with USBR or U.S. Army Corps of Engineers requirements.

- Borehole inclinometers will be installed and monitored in accordance with ASTM D-6230 (Test Method for Monitoring Ground Movement Using Probe-Type Inclinometers).

- Monitoring wells will be protected in accordance with ASTM D-5787 (Standard Practice for Monitoring Well Protection).

- Groundwater conditions in the Surface Mine and Waste Rock Pile Areas should be evaluated in accordance with ASTM D-5979 (Standard Guide for Conceptualization and Characterization of Ground-Water Systems).

5.6 Quality Control

Quality control will be performed on a continuous basis by site personnel as work progress in the field. Field record books will be maintained as necessary and field logs will be maintained and copied daily to eliminate the possibility of lost data. Approximately 5 to 10 percent additional samples will be collected in the field, beyond those specified, for later testing if test results appear to be in error.

Samples will be handled, packaged, labeled and shipped to the testing laboratory in accordance with accepted ASTM and EPA standards. All testing by the laboratory will be performed in accordance with accepted ASTM standards including all required data and information reporting required by the standards.

Field logs of borings and test pits will be reviewed and corrected as necessary based on the laboratory testing. The geotechnical report will be developed by consultants for W.R. Grace and reviewed by the various parties involved in the program.

Surveying for location and elevation of borings and test pits will be performed in accordance with accepted survey standards of the American Congress on Surveying and Mapping (ACSM) and the National Society of Professional Surveyor (NSPS).

Table 5-1: Groundwater Level Data

Description	Location	Sample Date	Water Level Elevation (ft)	Water Level (ft bgs)	Notes
Kootenai Development Impoundment Dam Piezometers (PVC Open-Tube, except P-O)	A8	2/20/2009	2797.8	8.2	
		1/15/2009	2797.7	8.3	
		12/1/2008	2797.8	8.2	
		10/30/2008	2797.8	8.2	
		10/2/2008	2797.9	8.1	
		8/8/2008	2799.0	7.0	
		7/3/2008	2801.3	4.6	
		6/3/2008	2803.0	2.9	
		5/20/2008	2803.3	2.7	
		5/16/2008	2802.1	3.9	
		4/23/2008	2798.4	7.6	
		3/10/2008	2797.6	8.4	
		5/8/2007	2800.7	5.2	
	P-0	-	-	-	Not Functional
	P1	5/8/2007	-	-	Dry
	P2	2/20/2009	2722.3	119.9	
		1/15/2009	2721.8	120.4	
		12/1/2008	2721.6	120.6	
		10/30/2008	2723.1	119.2	
		10/2/2008	2724.3	117.9	
		8/8/2008	2726.5	115.8	
		7/3/2008	2736.8	105.4	
		6/3/2008	2754.7	87.5	
		5/20/2008	2751.8	90.5	
		5/16/2008	2750.9	91.3	
		4/23/2008	2727.8	114.4	
		3/10/2008	2722.6	119.7	
		2/7/2008	2722.1	120.1	
		5/8/2007	2734.6	107.6	
	P3	5/8/2007	-	-	Dry
	P4	5/8/2007	2746.2	105.2	
	P5	5/8/2007	2763.8	103.6	
	PM1	2/20/2009	2757.6	53.7	
		1/15/2009	2757.4	53.9	
		12/1/2008	2757.4	53.9	
		10/30/2008	2757.4	53.9	
		10/2/2008	2757.4	53.9	
		8/8/2008	2758.2	53.1	
		7/3/2008	2761.6	49.7	
		6/3/2008	2762.9	48.4	
		5/20/2008	2763.1	48.2	
		5/16/2008	2764.9	46.5	
		4/23/2008	2761.1	50.2	
		3/10/2008	2759.8	51.5	
		5/8/2007	2761.7	49.6	

Table 5-1 Continued

Description	Location	Sample Date	Water Level Elevation (ft)	Water Level (ft bgs)	Notes
Kootenai Development Impoundment Dam Piezometers (PVC) Continued	PM2	2/20/2009	2734.0	103.7	
		1/15/2009	2733.7	104.1	
		12/1/2008	2733.7	104.1	
		10/30/2008	2733.9	103.9	
		8/8/2008	2736.7	101.1	
		7/3/2008	2740.3	97.5	
		6/3/2008	2747.1	90.7	
		5/20/2008	2749.8	88.0	
		5/16/2008	2749.4	88.4	
		4/23/2008	2736.7	101.1	
		3/10/2008	2734.3	103.5	
		5/8/2007	2741.6	96.2	
	PM3	5/8/2007	2767.5	51.6	
	PM4	5/8/2007	-	-	Dry
	PM5	5/8/2007	-	-	Dry
	PM6	5/8/2007	-	-	Dry
"CCC Well" in Carney Creek drainage, upstream of pond below fine tailings	Well A	7/22/2008	3349.6	1.8	
		9/29/2008	3351.0	0.4	
		10/1/2007	3348.1	3.3	
In clearing across small creek south of tailings dam, upstream of Watergate	Well C	7/22/2008	2764.6	22.8	
		9/29/2008	2763.3	24.1	
		10/1/2007	2763.4	24.1	
In pump house above (east of) tailings pond dam, potable supply well. Well log dated 11/28	Well D	7/23/2008	3583.6	241.5	
		9/30/2008	3581.2	243.9	
		10/1/2007	3579.6	245.5	
		2/25/1986	3584.2	240.9	
"MW-1" just off road on broad top level, ESE of pump house	Well E	7/23/2008	3789.0	172.6	
		9/30/2008	3770.6	191.1	
		10/1/2007	3883.4	78.3	
		9/22/2000	3771.2	190.5	
2-inch PVC well on edge of slope above (north) of Carney Cr.	Well F	10/1/2007	3406.4	213.9	Poor Condition
West of Mine "MW-2"	Well H	7/24/2008	3281.3	59.9	
		9/30/2008	-	-	Dry
		10/4/2000	3287.4	53.8	
Zinner Well Study Area 1	Z26	7/1/1981	-	-	no wl
	Z27	7/1/1981	3758.0	-2.0	Artesian
	Z28	7/1/1981	3755.0	-12.5	Artesian
	Z29	7/1/1981	-	-	no wl
	Z30	7/1/1981	3751.0	-9.7	Artesian
	Z31	7/1/1981	3748.0	-2.0	Artesian
	Z32	7/1/1981	3744.0	8.6	Artesian
	Z33	7/1/1981	3741.0	4.6	Artesian
	Z34	7/1/1981	3740.0	1.5	Artesian
	Z35	7/1/1981	3734.0	14.7	

Table 5-1 Continued

Description	Location	Sample Date	Water Level Elevation (ft)	Water Level (ft bgs)	Notes
Zinner Well Study Area 1 Continued	Z36	7/1/1981	3735.0	17.6	
	Z37	7/1/1981	3666.0	80.6	
	Z38	7/1/1981	-	-	no wl
	Z39	7/1/1981	-	-	no wl
	Z44	7/1/1981	3615.0	95.3	
	Z45	7/1/1981	-	-	no wl
	Z46	7/1/1981	3714.0	-4.7	Artesian
	Z47	7/1/1981	3713.0	-9.2	Artesian
	Z48	7/1/1981	3705.0	-16.6	
	Z49	7/1/1981	3710.0	-10.2	Artesian
	Z50	7/1/1981	3702.0	-1.5	
	Z51	7/1/1981	3696.0	5.7	
	Z52	7/1/1981	3649.0	56.9	
	Z53	7/1/1981	3627.0	84.8	
	Z54	7/1/1981	3550.0	158.2	
	Z55	7/1/1981	3545.0	161.2	
	Z56	7/1/1981	-	-	no wl
	Z57	7/1/1981	3552.0	139.4	
	Z58	7/1/1981	-	-	
	Z59	7/1/1981	-	-	no wl
	Z60	7/1/1981	3516.0	167.2	
	Z62	7/1/1981	-	-	no wl
	Z63	7/1/1981	3544.0	143.9	
	Z64	7/1/1981	3618.0	68.3	
	Z65	7/1/1981	-	-	no wl
	Z66	7/1/1981	3675.0	2.8	
	Z67	7/1/1981	-	-	
	Z68	7/1/1981	3664.0	70.9	
	Z69	7/1/1981	3712.0	12.9	
	Z70	7/1/1981	3718.0	1.6	Artesian
	Z71	7/1/1981	3732.0	-20.3	Artesian
	Z72	7/1/1981	3728.0	-2.7	
Zinner Well Study Area 2	Z83	7/1/1981	-	-	no static wl
	Z84	7/1/1981	3417.0	63.2	
	Z85	7/1/1981	3414.0	70.0	
	Z86	7/1/1981	3348.0	122.2	
	Z87	7/1/1981	3334.0	123.7	
	Z88	7/1/1981	3338.0	123.0	
	Z89	7/1/1981	3351.0	89.0	
	Z90	7/1/1981	3445.0	17.7	
	Z91	7/1/1981	-	-	no static wl
	Z92	7/1/1981	3445.0	82.3	
	Z93	7/1/1981	3446.0	78.2	
	Z94	7/1/1981	3456.0	74.9	
	Z95	7/1/1981	3479.0	49.0	

Table 5-1 Continued

Description	Location	Sample Date	Water Level Elevation (ft)	Water Level (ft bgs)	Notes
Zinner Well Study Area 2 Continued	Z96	7/1/1981	3485.0	41.3	
	Z97	7/1/1981	-	-	no static wl
	Z98	7/1/1981	-	-	no static wl
	Z99	7/1/1981	3467.0	76.4	
Zinner Well Study Area 3	Z73	7/1/1981	3418.0	72.6	
	Z74	7/1/1981	3407.0	85.0	
	Z75	7/1/1981	3410.0	81.7	
	Z76	7/1/1981	3443.0	46.4	
	Z78	7/1/1981	-	-	no static wl
	Z79	7/1/1981	3466.0	93.1	
	Z80	7/1/1981	-	-	no static wl
	Z81	7/1/1981	-	-	no static wl
	Z82	7/1/1981	-	-	no static wl
	Z100	7/1/1981	-	-	no static wl
	Z101	7/1/1981	3506.0	145.9	
	Z102	7/1/1981	-	-	no static wl
	Z103	7/1/1981	3529.0	111.7	
	Z104	7/1/1981	-	-	no static wl
	Z105	7/1/1981	3531.0	91.1	
	Z106	7/1/1981	-	-	no static wl
	Z107	7/1/1981	3540.0	99.0	
	Z108	7/1/1981	-	-	no static wl
	Z109	7/1/1981	3544.0	113.4	
	Z110	7/1/1981	-	-	no static wl
	Z111	7/1/1981	3556.0	100.5	
	Z112	7/1/1981	3562.0	90.1	
	Z113	7/1/1981	3608.0	44.0	
	Z116	7/1/1981	3418.0	96.4	
	Z117	7/1/1981	3458.0	62.2	
	Z118	7/1/1981	3410.0	111.3	
	Z119	7/1/1981	3458.0	69.4	
	Z120	7/1/1981	3460.0	69.5	
	Z121	7/1/1981	3460.0	67.1	
	Z122	7/1/1981	-	-	no static wl
	Z123	7/1/1981	-	-	no static wl
	Z124	7/1/1981	-	-	no static wl
Carney Creek Seeps/Springs	CCS-1	6/28/2008	3472.5	0.0	Seep
	CCS-11	6/28/2008	3723.5	0.0	Spring
	CCS-14	6/28/2008	3761.1	0.0	Spring
	CCS-16	6/28/2008	3676.3	0.0	Seep
	CCS-6	6/28/2008	3285.2	0.0	Seep
	CCS-8	6/28/2008	3254.1	0.0	Seep
	CCS-9	6/28/2008	3005.7	0.0	Seep

Table 5-2: Summary of Geotechnical Investigations

Boring, Test Pit or Item ID	Bulk Samples	Undisturbed Samples	Index Tests	Moisture-Density Tests	Compaction Tests	Strength Tests	Rock Durability Tests	Organic Content Tests	Piezometers or Monitoring Well	Comment
TSF-B1	2-3	2	3-4	2-3	1	1 TX	1-2 RQDs		Install New Piezo. & Transducer & Data Logger	At Max. Dam Section
TSF CPT-1 to 3	----	------	------	------	------	-------	-------	------	------	Std. CPT Rpt
TSF Existing Piezo. P-0	------	------	------	------	-------	------	-------	------	Install VW Piezo.	Repair Piezo.;Add Data Logger
Existing P-2 and PM-2									Install Transducers	Data Loggers
CTP-TP1	1-2	1	1-2	1	--------	------	-------	1	Install New Piezo.	----
CTP-TP2	1-2	1	1-2	1	------	------	-----	1	Install New Piezo.	----
CTP-TP4	1-2	1	1	1	------	------	------	1	-------	-----
CTP Geologic Recon.	------	------	------	------	------	-----	------	----	-------	Possible SM/Inclin
SMA-TP2	1	-	1				-------	1	-------	-------
WRP-B1	2-3	2-3	2-3	1-2	1	1-TX	------	------	-------	Install Inclinometer
WRP-B2	1-2	1-2	1-2	1-2	-----	------	------	------	------	New MW
WRP-B3	1-2	1-2	1-2	1-2	------	-------	--------		--------	New MW
WRP-TP1	1-2	1	1	1	-------	------	1 F-T or S-D	1	-------	---
WRP-TP3	1-2	1	1	1	-----	------		1		----
WRP-TP4	1-2	1	1	1	1	1-DS	1 S-D or F-T	-----	------	----

Notes:
TSF denotes Tailing Storage Facility
CPT denotes Cone Penetrometer Test
F-T denotes Freeze-Thaw Test
CTP denotes Coarse Tailing Pile
WRP denotes Waste Rock Pile
SMA denotes Surface Mine Area
DS denotes Direct Shear Test.
VW denotes Vibrating Wire Piezometers
S-D denotes Slake-Durability Test
TX denotes Triaxial Shear Test

Settlement Monuments at TSF and WRP areas not shown and existing MWs not indicated although water level measurements required from all existing MWs

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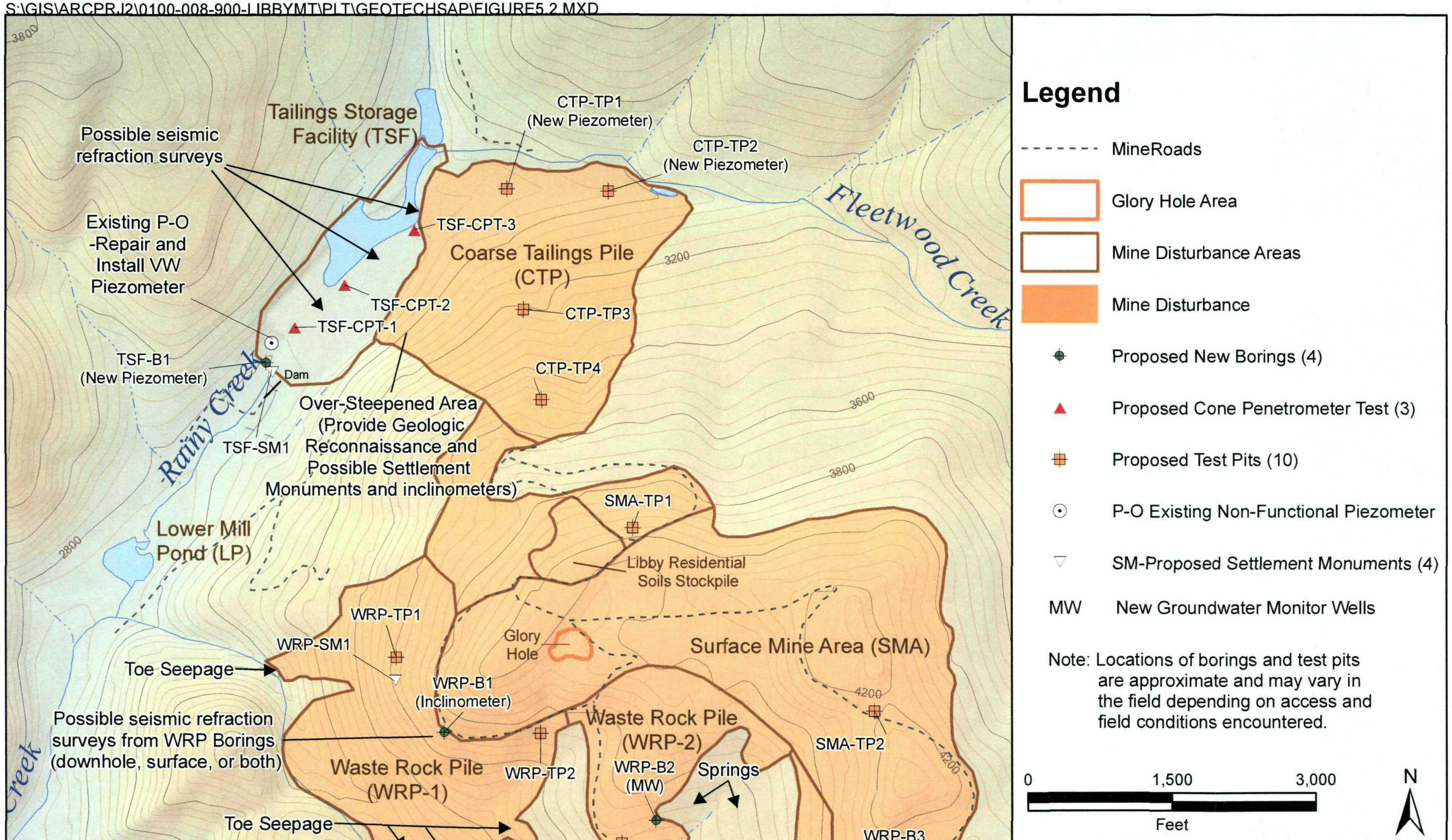
S:\GIS\ARCPRJ2\0100-008-900-LIBBYMT\PLT\GEOTECHSAP\FIGURE5.1.MXD

S:\GIS\ARCPRJ2\0100-008-900-LIBBYMT\PLT\GEOTECHSAP\FIGURE5.2.MXD



LIBBY MONTANA SUPERFUND SITE
OPERABLE UNIT 3

FIGURE 5-2

PHASE III SAP GEOTECHNICAL EXPLORATIONS FOR TAILINGS & MINE AREA

DATE: JUL 22, 2010
BY: SLG
FOR: JHR

FORMATION ENVIRONMENTAL

ATTACHMENT 4

CONSISTENCY :
- RAINY CREEK, NOT RAINEY
- REFER TO TAILINGS IMPOUNDMENT DAM AS KDID

5.0 OTHER DATA NEEDS FOR RI/FS

Additional geotechnical data are needed to support characterization of site conditions and evaluation of remedial alternatives in the FS. The long term effectiveness of the No Action alternative will require information to assess the stability of mine features and their potential to release materials into the environment. Potential source areas to be investigated are identified as follows:

- REPORTEDLY

- Tailing Storage Facility;
- Coarse Tailing Pile;
- Surface Mine Area; and
- Waste Rock Piles.

This section addresses the data requirements, data quality assessment, data quality objectives, sampling design, analytical requirements and quality control that are needed for the required geotechnical data at OU3.

5.1 Data Requirements

This section presents the background information necessary to assess the geotechnical engineering data requirements for OU3 RI/FS. ~~This information is developed~~ ~~from various sources.~~

DESCRIPTION OF
Tailing Storage Facility

The Tailing Storage Facility on Rainy Creek is impounded by a high-hazard, 135-feet high dam (127 ft reported by Harding, Lawson and Associates [HLA], 1992), initially constructed in 1971 with a 50-feet high starter dam (Schafer and Assoc., 1992).. The dam is classified as high hazard due to its size and presence of hazardous constituents. The tailing dam is also known more recently as the Kootenai Development Impoundment Dam (Billmayer Engineering, Inc. 2007a) and previously as the W.R. Grace Vermiculite Tailings Impoundment or the W.R. Grace Dam, Rainy Creek, Montana (Schafer and Associates, 1992 and HLA, 1992). Most recently the tailing dam has been called the Kootenai Development Impoundment Dam (KDID) in the 2008-2009 periodic Owners' inspection report (Billmayer & Hafferman, 2009)

SHOULD ACREAGE ONLY INCLUDE AREA COVERED BY TAILINGS IN WHICH CASE IT SHOULD NOT VARY - ?

The Tailing Storage Facility covers an area of approximately 53 acres (75 acres at maximum flood pool), a portion of which contains open water area of several acres depending upon the inflow to the impoundment. The volume of impounded water at the emergency spillway crest is approximately 937 acre-feet and the volume at the dam crest is approximately 1,302 acre-feet (Schafer and Assoc., 1992). The impounded water is typically approximately 500 feet upstream of the tailings dam; however, during extreme flood events water could be impounded adjacent to

(LOCATED —— FT UPSTREAM OF THE TAILINGS IMPOUNDMENT DAM)

THE DURING SPRING RUN-OFF SURFACE WATER SAMPLING PROGRAM ~~CONDUCT~~ (~~BH~~ ELEMENT — OF PHASE II A OF THE RI FOR OU-3 (EPA 2008)) FOR ___ WEEKS

the dam. The impounded water discharged over the spillway during the 2008 spring runoff period, and typically discharges during normal precipitation years.

WAS OBSERVED TO DISCHARGE

(A) DURING THE DEVELOPMENT OF THIS GEOTECHNICAL WORK PLAN

The original tailing dam designer was HLA and Bovay Engineers, Inc. which performed the design in 1971 (Bovay Engineers, Inc. and Harding, Lawson Associates, 1971). Several drawings were reviewed from this design including design drawings from 1979 (W.R. Grace Co., 1979). The original design drawings indicate a 50-feet high starter dam with 2:1 side slopes, a 40-feet wide crest and a downstream chimney drain. THE DRAWINGS SHOW The drainage system ~~is shown~~ starting upstream from the starter dam and extending along the foundation through each dam raise. Perforated cross drains are shown in the foundation for the Phase 1 and Phase 3 downstream embankment raises. (B) Initial embankment materials are shown as "Zone 3 and abutment excavation" material, although no further description of these materials is given to identify if they are silty gravels, sands or clays etc. (A) The drawings indicate three downstream raises of approximately 10 to 25 feet (Phases 1, 3 and 5) and two smaller centerline crest raises of approximately 5 to 10 feet (Phases 2 and 4). The fourth centerline raise to El. 2900 ft AMSL occurred in 1979 (Shafer and Associates, 1992). The fifth raise to El. 2926 ft AMSL is shown as a downstream raise apparently performed in 1981 (Billmayer & Hafferman, 2009). The downstream slope is shown as 2:1 with two benches each 10 feet wide. The centerline of the starter dam is shown as approximately 100-feet upstream from the 1979 dam crest centerline. The maximum design height of the embankment appears to have been 200 feet with downstream raises (Billmayer & Hafferman, 2009). Therefore, the current embankment height is approximately 67 percent of the final intended design height. 135 FT,

The crest length is approximately 1,100 feet. A ~~a~~ concrete box culvert with principal spillway discharge is located on the left (east) abutment and an emergency spillway at a higher elevation is located on the right (west) abutment. The principal spillway has an outfall to Rainey Creek below the dam and the emergency spillway does not appear to have an outfall to the creek. SP

~~It appears from~~ original drawings SHOW that foundation stripping up to about 5 feet in the valley bottom was performed to remove surface silts and that the abutments were stripped and benched. Original gravel blanket drains are shown in the design with perforated pipes to the downstream face, which were extended and added to during subsequent raises. Coarse tailing materials from the over-steepened area of the Coarse Tailing Pile were reportedly used in one or more of the dam raises. However, it is not clear where the coarse tailings might have been used in the embankment. Materials used for each embankment raise, whether centerline or downstream, are not defined in the available drawings.

OVER-STEEPENED AREA NOT INTRODUCED IN TEXT YET

AVAILABLE DATA

14 A series of eight boreholes to maximum depths of about 55 feet below original ground surface and 4 test pits to a maximum of about 17 feet below ground surface (ft bgs) are shown on the 1971 drawings in the vicinity of the starter dam and downstream of the starter dam. The borings do not have SPT values and do not indicate consistency of materials (loose, dense, very dense etc.). (C) Silt depths of up to 5 feet are indicated underlain by gravelly sand to sandy gravel of unknown consistency. The pyroxenite bedrock underneath the dam appears to be approximately 26-36 ft bgs. Bedrock ~~on the right side~~ near the west abutment appears to be deeper, about 40 to

SHOULD WE INCLUDE A DRAWING?

SPELL OUT

BORE HOLE ~~AND TEST PIT~~ DATA INDICATE

45 ft bgs. Bedrock on the ~~right~~ WEST abutment appears to be about 12 to 18 ft bgs and on the ~~left~~ EAST abutment appears to vary from about 8 to 12 ft bgs. The rock is only about 1 ft bgs further up the ~~left~~ EAST abutment area. A zone of silts and clays is indicated at depths of approximately 19 to 26 ft bgs and 35 to 37 ft bgs near the ~~right~~ WEST abutment downstream toe. It is not indicated if these are soft zones. No test pits or borings are indicated in the impoundment area.

(c) LIMITATIONS:

Tailings consist of interbedded layers of soft to stiff elastic silt (60%) and loose to medium dense poorly-graded sands and silty sand (40%) with mica and pyrite flakes. Based on two borings in the east side of the impoundment, the maximum thickness of tailings in the impoundment is approximately 70 to 75 feet (HLA, 1992). Confirmation of these depths and estimation of depth variations over the impoundment area, particularly further upstream, have not been performed. The loose silty sand tailing materials are reported to have liquefaction potential during seismic events (HLA, 1992).

NEED DRAWING?

Embankment soils reportedly consist of dense to very dense, well graded silty sands. The overall downstream embankment slope is shown on stability models to be approximately 4(horizontal):1(vertical), although existing slopes appear to be steeper than this. The ~~right~~ WEST abutment is underlain by a thick blanket of glacial outwash and till from a few feet to 40 feet thick. The ~~left~~ EAST abutment slope is blanketed by a relatively thin mantle of slope debris and remnants of a lateral moraine near the base of the canyon slope with an intermediate 4-feet thick zone of highly permeable, relatively clean sand. Natural foundation soils consist primarily of dense to very dense poorly-graded gravels, dense to very dense poorly-graded sands and moderately hard, friable pyroxenite bedrock with abundant magnetite and pyrite (HLA, 1992).

29 ~~30~~ FT BELOW DAM CREST OF 2926

IN DESCRIPTION?

The surface discharge from the tailing impoundment discharges through a reinforced concrete principal spillway at a crest elevation of 2,897 ft AMSL located on the left (east) abutment. An inlet channel presently extends from the pond several hundred feet upstream from the dam crest to the principal spillway inlet. The principal spillway consists of an 8-feet wide by 4-feet high concrete box culvert approximately 169-feet long and an 8-feet wide by 3-feet high concrete discharge channel approximately 965-feet long with a concrete and riprap outfall. The principal spillway has a reported full-channel discharge capacity of 731 cubic feet per second (cfs) with the water surface at the dam crest. The concrete structures are reported to be partially cracked with some rocks and debris near the inlet.

REPORTED WHERE?

WHERE? CHECK MOST RECENT REPORTS - THEY INDICATE REPAIRS HAVE BEEN MADE. CHANGE TEXT TO: THE MOST RECENT REPAIRS, PERFORMED IN (DATE) CONSISTED OF —

The principal spillway and an emergency spillway, located on the right (east) abutment, are reportedly designed for one-half of the probable maximum flood (1/2-PMF; Schafer and Assoc., 1992). The peak inflow from the total Rainy Creek and Fleetwood Creek upstream drainage area (9.4 square miles) for the ½-PMF event was computed to be 5,838 cfs. The storage capacity of the impoundment was estimated to be approximately 1,302 acre-feet at the dam crest (Schafer and Assoc., 1992). Routing the ½-PMF flood hydrograph through the reservoir resulted in a peak discharge flow significantly lower than the peak inflow, and the present system of concrete principal spillway on one abutment and earth-riprap emergency spillway on the other abutment was recommended and constructed in the early 1990s. The emergency spillway is reported to be

BY?

4 FT BELOW DAM CREST

approximately 35-feet wide by 380-feet long with riprap erosion protection at an elevation of approximately 2,922 ft AMSL. The capacity of this emergency spillway with the water surface at the dam crest is reported to be 1,129 cfs (Billmayer & Hafferman, 2009). Thus, the combined discharge capacity of the principal and emergency spillways is approximately 1,860 cfs.

1129 CFS
+ 731 CFS
1860 CFS

HAVE TO MAKE THIS LINK

Recent risk-based analyses of the tailing dam concluded that the potential loss-of-life is 0.41 (Billmayer & Hafferman, 2009). Based on the current Montana spillway standards, the spillway design flow is therefore downgraded to an inflow design flood having a recurrence interval of 500 years. Analyses performed for this flood event determined the peak inflow from Rainy and Fleetwood Creeks to be 351 cfs utilizing USGS regression equations for Montana stream peak flows. This method provides an approximate method of determining peak flow with a standard error of prediction of approximately 67 to 79 percent. Therefore, based on this analysis, the existing ~~peak design flow~~ DISCHARGE CAPACITY of 1,860 cfs for the spillway system, which is based on the ½-PMF inflow, is well in excess of the latest ESTIMATE FOR peak inflow from the 500-year flood event. Analyses were performed assuming loss of upstream vegetation due to a forest fire with a ground cover of approximately 20 percent. The peak inflows for this condition were estimated to be approximately 851 cfs. Environmental risk analyses have not been performed for the tailing storage facility. WHAT DOES THIS MEAN?

COMPARED TO 5,838 CFS BASED ON 1/2 PMF

By comparison, previous studies performed using a hydrograph analysis with full forest vegetation conditions and an overall hydrologic Curve Number of 60, estimated the combined peak flow from the 100-year, 24-hour storm event in Rainy and Fleetwood Creeks to be approximately 460 cfs (Schafer and Assoc., 1992). ADD, SO WHAT'S THE CONCLUSION?

WE'VE GOT ESTIMATES OF PEAK INFLOWS THAT RANGE FROM 351 CFS ↓ 460 CFS ↓ 851 CFS ↓ 5838 CFS

BASED ON MEASUREMENTS TAKEN WHEN?

Several open-tube piezometers are located in and near the dam embankment, which indicate either dry conditions or relatively low water levels. The maximum phreatic surface is reported to be approximately 94 feet below the crest of the dam, or approximately 40 feet above the base of the dam. One piezometer (P-2) is reported to fluctuate several feet each year and up to a maximum of approximately 33 feet. A piezometer at the dam toe (A-8) indicates piezometric surfaces varying approximately 5.5 feet at that location. The peak of the highest phreatic water surface each year corresponds to the peak of the snowmelt/rain runoff in the area in the late spring. Only one piezometer is located within the tailing impoundment, which is reported to have not been measurable the last few dam inspections. This piezometer (P-O) consists of a 2-inch diameter PVC casing with two ¼-inch tubes inside, and appears to require compressed air for reading (Billmayer & Hafferman, 2009).

HOW MANY? TOE DRAINS — USE CONSISTENT TERMINOLOGY

HOW OFTEN?

A series of seepage-control pipes are located on the downstream embankment which have been maintained periodically (Billmayer, 2007a). The most recent dam inspections in December 2008 and January 2009 reported on each of the twelve seepage pipes exiting the downstream embankment (Billmayer & Hafferman, 2009). This report also discussed the various piezometers in and near the dam embankment. It was concluded that the drains and the phreatic surface indicated by the piezometers follow the yearly surface water flow fluctuations. It was

PEAK FLOW ESTIMATES FOR INFLOW FROM RAINY AND FLEETWOOD CREEKS

(1992) 1/2 PMF 5838 CFS
(2009) 500 YR FLOOD 351 CFS
500 YR FLOOD, 20% GROUNDCOVER 851 CFS
(1992) 100 YR, 24 HR STORM FULL FOREST VEGETATION 460 CFS

further concluded that the majority of the volume of stream flow upstream of the tailing impoundment infiltrates the tailings and subsequently reports to the toe drainage system. A portion of the surface flow also discharges through the principal spillway during the late spring most years.

~~CAN YOU~~ SAY WHY THIS REPORT WAS DONE? – I THINK IT WAS A REQ. OF THE 5-YR REVIEW OF THE DAT

A surface water, piezometer and toe drain monitoring report was prepared for the water year 2008-2009 (Billmayer & Hafferman, 2010a). This report indicated approximately 1,154 acre-feet of water flowed into the tailing facility from Rainey and Fleetwood Creeks during the period from October 2, 2008 to October 23, 2009. Approximately 88% of this water discharged through the toe drainage system, 10% discharged through the principal spillway and 2% was lost to groundwater recharge. Therefore, the great majority of the inflow to the tailing facility reports to the toe drainage system. The maximum capacity of the toe drainage system is approximately 1,800 gallons per minute (gpm) and the base flow varies from approximately 200 to 400 gpm. This report concluded that: "*lacking a means to currently cut off drain flow or bypass inflow, the entire stability of the KDID will depend on the ability of the drains to discharge the water that infiltrates the reservoir and upstream face of the embankment. Therefore, the safety of the KDID will depend solely on making sure that there is always full drain flow capacity*" (Billmayer & Hafferman, 2010b).

USE CONSISTENT REFERENCES

DEFINE

The February 2010 B&H report described periodic water level data from 4 of the 12 piezometers located at the tailing dam from the dam crest to the toe. Some of the existing piezometers are not functional and some of the piezometers closer to the abutments remain dry. A summary of these piezometer data and other groundwater well data throughout the site are presented on Table 5-1. The dam piezometer data indicate that the majority of the seepage from the impoundment intercepts the main toe drain system along the centerline of the dam near the maximum section. The data also indicate that inflow to the impoundment has an almost immediate effect on the drain flow and a slightly delayed effect on the phreatic surface through the embankment. The recent highest phreatic surface through the embankment remained below the maximum phreatic surface modeled in the 1992 stability analysis.

FROM WHAT PROGRAM?

UPDATE, I THINK THEY HAVE BEEN REPAIRED PLEASE CORRECT

Recent inspections of the interior of the twelve toe drains were performed using a video camera (Billmayer & Hafferman, 2010b). These inspections indicated that the toe drains are in fair to poor condition overall. Several of the drains were crushed or had gaps in the joints or other penetrations which allowed roots and moss in some of the drains and silt, sand and rock in other portions. One of the drains had turbid flow and another was transporting material out of the embankment. Inspections indicated that seepage flow was occurring outside some of the drain pipes and soft, wet areas are present near some drain outlets. Only the largest, central metal drain appeared to provide clear and unobstructed flow, although the pipe interior is corroded. This report recommended further investigations of the toe drain system. UPDATE

B&H (2010a) recommended that "*the active piezometers and possibly drain flow could be monitored with transducers those transducers could be wired to a transmitter sending out real* *time data. Real time piezometer and transducer data would provide the best opportunity to have real time records and real time warnings when changes in these readings indicate problems."* It was further recommended in this report that: "*a piezometer tube be installed adjacent to the spillway and that a transducer be placed in the piezometer and monitored for at least one if not two seasons.*" This report also recommended that a flow measuring flume be installed in Fleetwood Creek and that additional inflow data to the tailing impoundment be obtained from Rainey and Fleetwood Creeks. Such data would likely provide important information for long-term planning of remedial alternatives at the tailing impoundment. ? I THINK THIS CUTS DAVE

A groundwater monitoring well, Well C, is located downstream of the Tailing Storage Facility, approximately mid-way between the facility and the downstream Mill Pond. This is a 10-inch diameter well, which originally contained a pump, and is approximately 72-feet-deep. Groundwater levels in this well have varied from approximately 22 to 25 feet below ground surface.

THESE SEEM TO JUST BE BULLET POINTS OF INFORMATION CAN YOU PROVIDE SOME CONTEXT OR SIGNIFICANCE?

~~SEISMIC~~ STABILITY DURING SEISMIC EVENTS

Previous studies have concluded that the tailing embankment is stable during static and seismic conditions with acceptable deformations reported for an analysis assuming a maximum credible earthquake producing a horizontal ground acceleration of 0.30g (HLA, 1992). These analyses were based on the state of Montana standards prior to development of the new Montana Dam Safety Standards for High-Hazard Dams. Previous analyses appear to have utilized two-dimensional models in 1992. Recent stability analyses, with updated seismicity conditions, do not appear to have been performed for the structure. Finite element analyses of stress conditions utilizing state-of-the-art models, do not appear to have been performed for the dam and foundation. WHAT YEAR ?

HAVE NOT BEEN

Seepage through the dam has been identified as a potential long-term stability concern, particularly if the impounded water is adjacent to the dam. A levee was recommended in the 1992 HLA study to be located approximately 500 feet upstream from the dam crest to prevent the pond from reaching the dam; however, the levee was not constructed. THIS IS OUT OF SEQUENCE

The Draft Environmental Assessment for the site (Montana Department of State Lands, 1992) identified a number of concerns with a full diversion of Rainy Creek around the Tailing Storage Facility including the following: *"The full diversion alternate increases the potential for failure, and decreases the safety of the system----Stability of the structure in a massive flood condition would be problematic---The channels carrying the diverted flows would be very large, and inherently less stable than smaller channels, particularly when constructed in the side of a hill as they would be in this case. From a hydrologic and geotechnical standpoint, any channel, natural or constructed, located above the low point in a drainage is generally not considered to provide good long-term service...Should diversion channels become plugged, or the system fail for some other reason, the flood flows would quickly breach the diversions and enter the impoundment"*. This opinion was reiterated in the 1992 Schafer Engineering Analysis of Flood Routing Alternatives report.

MOVE TO C

It was noted in the February B&H report (2010a) that the tailing impoundment and dam structure, including the drainage system, are now decades old. It was further recommended in this report that: "*the feasibility of cutting off the drain flow, breaching the reservoir, by-passing the reservoir, or a combination of any or all or any other feasible means of decommissioning the KDID should continue to be investigated.*"

C

? Geotechnical data including boring logs and laboratory testing were developed for this tailings dam during the 1992 study. Long-term maintenance of this tailings dam, to be evaluated in the FS, will be based on existing and additional data related to the geotechnical characteristics, confirmation of depth and extent of the tailings in the impoundment.

CAN YOU EXPRESS THIS IN TERMS OF ONE OF THE NINE FS CRITERIA?

Coarse Tailing Pile

The Coarse Tailing Pile is located on the hillside east of the tailing impoundment and covers an area of approximately 140 acres. Based on topographic mapping, the total height of the Coarse Tailing Pile is approximately 700 feet and has side slopes of approximately 3:1 to 4:1. The Coarse Tailing Pile reportedly has had some reclamation procedures applied as discussed below. A small surface impoundment is located at the east toe of this pile covering an area of approximately 16,000 square feet (sf). Fleetwood Creek extends along the north toe of the Coarse Tailing Pile and storm flow events likely extend the floodplain over the toe of the Coarse Tailing Pile although specific hydrologic/hydraulic information was not identified for review.

INDICATE ON FIGURE 5-1

A portion of the Coarse Tailing Pile appears to be at the slopes of 2:1 to 4:1 and a portion, approximately 65 acres, is reported to be too steep or over-steepened. This over-steepened portion was reportedly the borrow source for a tailing dam raise although documentation of this activity has not been identified. The northwest portion of the Coarse Tailing Pile extends into the upstream portion of the tailing impoundment and may have stability concerns. BASED ON WHAT?

SO HOW IS IT REPORTED?

Groundwater level data beneath the Coarse Tailing Pile and in the surrounding areas were not available for review. Groundwater level data for a number of borings are presented in the 1982 Zinner Report, which included a zone near the upgradient portion of the Coarse Tailing Pile area extending approximately 2,500 feet to the east (Figure 5-1). These indicated groundwater levels varying from approximately 41 to 146 feet below ground surface in the summer of 1981 (Table 5-1). IS A TABLE THE BEST WAY TO SUMMARIZE THIS DATA?

The Coarse Tailing Pile has reportedly undergone reclamation work including run-on control, contouring for runoff control, seeding, and planting of trees (Ray, 1999). The existing reclamation work has not been reviewed as part of this data needs assessment. This reclamation work was, however, reviewed for bond release by the Montana Department of Environmental Quality (MDEQ, 1999a).

A portion of the Coarse Tailing Pile reportedly experienced snowmelt/rain runoff erosion in 2007. This area reportedly required an estimated 6,500 cubic yards (cy) of restoration fill from nearby waste rock and relocation of an under-road culvert which apparently caused the washout (Remedium, 2007). Information regarding implementation of this erosion restoration was not identified for review. FIND OUT IF THIS WAS DONE

By whom?

Geotechnical data for the Coarse Tailing Pile were not identified, other than anecdotal descriptions. Various issues were raised following bond release in 1999 including comparable stability and utility of reclaimed areas and levels of asbestos on the surface of reclaimed areas and potential for continuing release (MDEQ, 1999b). It appears that insufficient data exist to adequately assess the long-term stability of the over-steepened area near the Coarse Tailing Pile. Such data will include geologic reconnaissance and settlement monuments in the over-steepened area followed by inclinometers if determined to be necessary based on the reconnaissance and settlement study. Geotechnical index parameters will need to be obtained from test pit samples in the coarse tailing area to determine engineering characteristics and the test pits will assist in determining the volume of the coarse tailing pile. Groundwater data beneath the Coarse Tailing Pile do not exist and piezometers will need to be installed in the north portion of the area to assess groundwater conditions.

THIS SEEMS TO BELONG IN DQO SECTION (?)

Surface Mine Area

The Libby Mine site is located on top of a mountain that is part of the Rainy Creek Igneous Complex and is the upper portion of a hydrothermally altered igneous pyroxinite complex, which intruded into the Precambrian Belt Series rock. "Vermiculite Mountain" is generally a biotite/vermiculite deposit occurring in a pyroxenite matrix. Intrusions of syenite and pegmatite, which originated from a nearby synenite body, lie within the deposit. The vermiculite ore body at the Libby Mine contains various quantities of non-asbestos amphiboles as well as quantities of asbestiform or fibrous amphiboles. Reference? SP

The Surface Mine Area covers an area of approximately 270 acres at the top of the mountain. The disturbed area of the Surface Mine Area is contiguous with the mine waste rock piles immediately to the south. The former mill area was located just west of the Surface Mine Area and all associated facilities have been removed. INDICATE ON FIGURE 5-1 AS FORMER MILL AREA

AS PART OF RECLAMATION (?) (REF.?)

The Surface Mine Area includes the former "Glory Hole" which covers an area of approximately 15 acres southeast of the former mill area and adjacent to the Waste Rock Pile area. This was reportedly filled with miscellaneous mine waste debris (typical Class II landfill material), then covered and seeded as part of reclamation (Ray, 1999).

FORMER or HISTORICAL LOCATION OF MILL & SUPPORTING FACILITIES

The Surface Mine Area was reportedly reclaimed in the 1990s including regrading, seeding and planting (Ray, 1999). This area was inspected for bond release in 1999 by the MDEQ. Issues

Needs Better Description

remaining included levels of asbestos on reclaimed areas and water quality concerns related to potentially hazardous materials disposed in the Glory Hole and other areas (MDEQ, 1999b).

A stockpile of soil removed from residential yards in Libby is located on the Surface Mine Area as shown on Figure 5-2. The existing volume of this stockpile has not been identified.

Previous geologic and hydrogeologic investigations conducted in this area provide limited information regarding the local groundwater flow systems. The available groundwater monitoring wells, along with observed springs, are shown on Figure 5-1. Two wells were installed by WR Grace in 2000: one monitoring well was installed adjacent to the Glory Hole (MW-1; Well E in later reports) and the other monitoring well was installed near the west toe of the old waste dump (MW-2; W.R. Grace & Co., 2000; Well H in later reports). Another well, Well D, is located in an old pump house building that may have served as potable water source for the mine.

WHAT OLD WASTE DUMP?

Well E (MW-1) near the Glory Hole is a 2-inch PVC well screened from 235 to 250 feet bgs. The log of the monitoring well indicates approximately 4 feet of rock fill over approximately 16 feet of vermiculite with weathered pyroxenite below this to a depth of approximately 82 feet below ground surface. Biotite pyroxenite bedrock was identified from a depth of 82 feet to the bottom of the borehole at a depth of approximately 250 feet below ground surface. Groundwater was found at a depth of approximately 242 feet below ground surface and produced approximately 1 to 2 gallons per minute. Recent water-level measurements of groundwater in Well E (MW-1) indicate groundwater levels varying from approximately 78 to 190 feet below ground surface. Groundwater level data are presented on Table 5-1.

Well H (MW-2) has a total depth of approximately 90 feet and is believed to be a more recent well located near a haul road on the hillside west of the mine. The well is also a 2-inch PVC, screened from 60 to 70 feet bgs. The well log indicates topsoil containing vermiculite for the first 5 feet overlying gravelly sand to approximately 15 feet bgs. Loose, fine sand with mafic minerals and areas of vermiculite are present to the bottom of the well. During groundwater sampling in July 2008 the total depth of the well was measured at 71 feet. Groundwater was identified in Well H (MW-2) during well construction at approximately 56 feet bgs and was reportedly high in arsenic and lead (MDEQ, 2000), although such data were not identified. Monitoring data from Well H in 2008 indicate groundwater levels approximately 60 feet to greater than 71 feet (dry conditions) below ground surface.

Well D is located at the bottom of a 5-foot-diameter culvert that extends approximately 8 feet below and 2 feet above the surrounding ground surface. The well is 10 inches in diameter, is cased with steel, and is screened from 345 to 385 feet bgs. The well initially was drilled to a total depth of 405 feet, then backfilled to 385 feet. Measurements in July 2008 indicate that the total depth extends to approximately 378 feet bgs with a soft sediment bottom. The well log indicates fill material to a depth of 37 feet bgs overlying vermiculite to 157 feet bgs. Pyroxinite and biotite pyroxinite bedrock were indicated from 157 to 215 feet bgs with a zone of dike material consisting mainly of quartz until approximately 378 feet bgs. The last 5 feet (to a total depth of 405 feet bgs) of the boring contained vermiculite. Data collected during construction indicated water levels at approximately 240 feet bgs and production of around 30 gpm. Measurements of groundwater levels in 2008 indicated levels approximately 241 to 244 feet bgs.

No records of groundwater monitoring wells or water level data have been identified in the eastern portion of the Surface Mine Area.

NOT HELPFUL

An earlier geo-hydrologic study was performed in areas north and south of the mine area that included drilling of approximately 100 boreholes to depths up to approximately 170 feet below ground surface (Zinner, 1982). These boreholes were located in the area between Waste Rock Piles 2 and 3 south of the mine and in an area east of the coarse tailing pile north of the mine. The boreholes indicated overburden materials from near zero to a maximum of approximately 90 feet bgs. Vermiculite pyroxenite was found below the overburden in thicknesses varying from approximately 40 to 170 feet bgs. Biotite pyroxenite bedrock was found from approximately 40 to 190 feet bgs.

NOT HELPFUL

Groundwater levels in these boreholes varied from the ground surface, with artesian conditions in the area between the waste rock piles, to approximately 140 feet below ground surface. The twelve artesian boreholes produced approximately 1 to 2 gpm water flow with release of trapped gas. Two boreholes north of the mine produced water flows of up to approximately 20 gpm. It was theorized that *"the aquifer is probably the result of a permeable zone of sandy and gravelly till overlain by a less pervious till"* (Zinner, 1982 [Harding and Lawson, 1974]). Zinner theorized that *"the artesian conditions are thereby the results of the upper inclination of glacial deposits to the canyon head where recharge takes place"* (Zinner, 1982). Confirmation of these theories has not been made and the areal extent of such conditions has not been determined.

As part of the same study, two deep boreholes were drilled in the mine area: one was drilled to a depth of 900 feet through the 22nd mining level (Hole 130) and one was drilled to a depth of 970 feet north of the mine area (Hole 131). The first deep borehole in the mine area indicated approximately 10 feet of overburden with 15 feet of vermiculite underlain by biotite pyroxenite to the 900 foot depth. This deep borehole produced approximately 25 gpm at the 500-foot depth, approximately 350 to 500 gpm was produced from a depth of 700 feet and drilling was stopped at 900 feet as approximately 1,000 to 2,000 gpm were being discharged to the surface. The final water level was approximately 66 feet below ground surface indicating the water level was under piezometric conditions. Another deep borehole was reportedly drilled 200 feet from Hole 130 which was reported to be under artesian conditions discharging approximately 5 gpm (Zinner, 1982). The second deep borehole north of the mine (Hole 130) did not encounter strong water producing zones as did Hole 130, although approximately 25 gpm was reported at a depth of approximately 500 feet below ground surface. The location and logs of deep boreholes 130 and 131 were not provided in the Zinner report.

SHOULD INDICATE LOCATIONS OF THESE DEEP BOREHOLES ON FIG. 5-1

DOESN'T MAKE SENSE

CHECK w/ MWH

Anecdotal information regarding an underground mine beneath the Libby Surface Mine Area has not been confirmed. Information regarding such underground workings was not identified during this investigation.

? WHERE WAS ANECDOTAL INFO FROM?

No additional geotechnical data were identified for review from the Surface Mine Area.

Mine Waste Rock Piles

The mine Waste Rock Piles are located south of the surface mine and cover a total area of approximately 230 acres. The toes of the Waste Rock Piles extend southwest to Carney Creek in some locations and the side slopes appear to be roughly at the angle of repose, and some contouring has reportedly been performed. The Waste Rock Piles consist of three major piles south and southeast of the former mill. For the purposes of this investigation, the larger Waste Rock Pile located to the south of the former mill site is designated WRP-1, the middle pile is designated WRP-2 and the southeast pile is designated WRP-3.

REPETITIVE ?

REPORTED WHERE ?

Topographic maps indicate that the largest WRP-1 has a total height in excess of 850 feet on the west side and has an overall slope of approximately 2:1 with haul roads and benches. Portions of the east side of WRP-1 and WPR-2 and WRP-2 have heights of approximately 150 to 200 feet at side slopes varying from 1.2: to 1.4:1. The existing hillside slopes vary from approximately 2.5:1 to 2.8:1. The topographic maps and aerial views (Google, 2010) indicate areas of gully erosion from the waste rock piles.

A small waste debris area, covering approximately 3 to 4 acres was located southwest of the mill. It was reported that miscellaneous debris (including drums) from this smaller Waste Rock Pile was disposed in an excavated area southwest of the mill site approximately 800 feet east of the Lower Pond (Ray Engineering, 1995). Water samples were reportedly obtained during reclamation of the small waste debris area but were not identified for review. This 1995 report also indicated movement of mine waste on the hillside thought to be caused by seepage from a spring and local areas of impounded water.

A land farm was reportedly developed for treatment of wastes from a leaking underground storage tank at or near the mill site. Information and data for this land farm treatment facility were not identified for review.

The Waste Rock Piles are reported to have undergone reclamation activities in the 1990s similar to the Coarse Tailing Pile and Surface Mine Area although the degree of reclamation is not known. A landslide area at one of the Waste Rock Piles covering approximately 45 acres exposed an old landfill in the 1990s, which was apparently reclaimed and the landfill debris was relocated elsewhere. The MDEQ reported that the landslide area had dried out and appeared to have stabilized (MDEQ, 1999a). However, hillside springs may re-appear at various locations depending upon snowpack and other factors. Decomposing vermiculite is typically a very weak material and its presence within the waste rock piles would tend to weaken the overall structures, particularly over time.

As discussed above, the Zinner report indicates artesian conditions in several of the boreholes between WRP-2 and WRP-3. It is not known how this artesian groundwater condition affects the stability of the waste rock piles. The Zinner report observed that the *"load created by the waste dumps and their impedance of water flow has created instability in the surrounding slopes and in the valley bottom"* (Zinner, 1982). Several springs have been located on the hillside between WRP-2 and WRP-3, which appear to confirm the presence of artesian conditions in this vicinity.

Well F was identified at the top of WRP-1, which is presently in poor condition. One groundwater data point was available for this well in October 2007 which indicated a groundwater level approximately 214 feet below ground surface. Well F is reportedly not in service at this time due to poor conditions of the well, and it is believed Well H, located approximately 2000 feet northwest of Well F, may be a suitable surrogate (Phase II SAP part B).

One groundwater monitoring well, Well A, is located just north of Carney Creek below WRP-1 and WRP-2. Well A indicates groundwater varying from less than one foot below ground surface during portions of the year to approximately 3 feet bgs.

A 1992 environmental assessment determined that *"the waste rock dump has inherent stability problems due to the structure of the ore and waste rock. The dump is currently standing at the angle of repose (1.25 to 1.5:1)....As a result of mass wasting, the waste rock dump toe has encroached on the Carney Creek stream channel. The slumping of waste rock has forced the creek to cut a new channel through the waste rock that has rolled to the bottom of the drainage in the end dumping process used to form the waste rock dump"* (Montana Department of State Lands, 1992).

Geotechnical data for the Waste Rock Piles were not identified and it appears there are insufficient data to assess the long-term stability of the facilities in the FS. Such data needed for analysis will include bulk samples for index parameters, compaction characteristics and strength parameters. Investigations will include test pits and geotechnical borings. Should be in DQO's

5.2 Data Quality Assessment

This data quality assessment includes a review of the identified engineering data for the Tailing Storage Facility, ~~which~~ primarily ~~includes~~ includes data for the impoundment dam related to stability and safety, ~~and~~ for the Surface Mine Area, w~~hich includes~~ limited monitoring well data, ~~Limited engineering data quality assessment is included for~~ the Coarse Tailing Pile, and the Waste Rock Pile ~~areas based on~~ very limited data adjacent to the areas. Areas

Tailing Storage Facility

CAN WE DEFINE PERIOD?

WHAT DATA IS AVAILABLE FOR THESE INSPECTIONS? HOW MANY INSPECTIONS?

(A) Because of the high-hazard rating of the tailing impoundment dam, geotechnical stability and hydrologic reports were completed for the facility in 1992 and periodic safety inspections have been performed since that time. Periodic safety inspections have found the structure to be safe with the implementation of additional maintenance measures associated with the downstream drainage system and with the addition of a reinforced concrete box culvert outlet through the left abutment and concrete discharge flume and chute downstream of the dam.

TOE DRAIN SYSTEM? WHERE?

(B) The geotechnical report completed in 1992 included 10 geotechnical borings to depths ranging from approximately 22.5 to 77 feet below ground surface (ft bgs). The soils were classified in accordance with American Society of Testing and Materials (ASTM) Standard D-2487 and visual-manual procedures were performed in accordance with ASTM D-2488. Standard Penetration Tests (SPTs) were performed in the borings in accordance with ASTM D-1586. Selected disturbed and undisturbed soil samples were tested for moisture content, dry density, Atterberg Limits, gradation, percent passing the No. 200 sieve, unconsolidated-undrained triaxial shear strength, consolidation and compaction characteristics. Although the testing procedures were not reviewed in detail, they were reportedly performed in accordance with established ASTM procedures.

MOVE TO (A) SHOULD BE 1st PARAGRAPH

The original design OF THE KDID in 1971 included 8 geotechnical borings and 14 test pits in the vicinity of the starter dam and downstream of the proposed dam embankment. No explorations were performed upstream in the impoundment area. The borings determined the depth to bedrock and the test pits indicated near surface conditions. Standard penetration data were not reported for the borings and the general subsurface conditions were described from the boring and test pits logs presented on the design drawings. It is not known what quality control procedures were utilized IN the sampling and analysis of subsurface materials.

WHAT TIME OF YEAR WERE SAFETY INSPECTIONS DONE?

Twelve piezometers at the tailing dam have WERE been monitored during the periodic safety inspections. All of these piezometers was monitored in the 2007, 2008-2009 and 2010 AND RESULTS ARE PROVIDED IN inspection reports (Billmayer & Hafferman, 2009 and 2010a). One additional piezometer not measured is apparently located in the impoundment area approximately 300 feet northeast of the dam crest. Annual monitoring of the piezometers have reportedly found the phreatic surface in the dam to be relatively low, with a maximum height of approximately 3 to 4 feet above the dam foundation (HLA, 1992 and Billmayer, 2007a). Seven of the thirteen piezometers monitored contained water during the 2007 annual inspection and the latest inspection reported similar conditions. Real-time piezometric data for the dam has not been performed because transducers and data loggers have not been utilized in the open-tube piezometers.

The 2007 inspection report concluded that the dam was in good to excellent condition and that no significant structural or maintenance concerns were found that would require immediate action (Billmayer, 2007a). The emergency action plan, operational plan, routine maintenance plan and piezometer monitoring logs were reported to be up-to-date and effectively addressed the structure and its components. The annual dam safety inspections have reportedly been approved by the Dam Safety Program of the Montana Department of Natural Resources (DNRC).

TOE DRAINS

The 2007 dam inspection report recommended cleaning the seepage outlet drains and performing minor maintenance work on the dam and concrete box culvert and chute spillway, some of which was described in a Montana 310 permit application (Billmayer, 2007b). Some of this work has apparently been performed and recent photographs of the inside of some drain pipes indicate some corrosion and deterioration (Billmayer & Hafferman, 2009). Long-term effectiveness of the existing dam drainage system has not been performed and is suspect due to the corrosion of some drain pipes. The most recent toe drain inspection report (Billmayer, 2010b) indicates that the toe drainage system is in generally fair to poor condition and the report recommended further field investigations of this system.

The 2007 dam inspection report also recommended that a review of bank stability and seismic stability be performed (Billmayer, 2007a). Documentation of this review has not been identified. The 2007 inspection report also recommended that preparation for the 5-year operational permit renewal inspection be conducted no later than the fall of 2008. These recommendations included: 1) development of a complete catalog of all available documentation and reports for the tailing dam, 2) a complete review of the stability analysis based on the latest piezometer data, and 3) a review of the seismic stability of the embankment based on the new Montana Dam Safety Seismic standards for high-hazard dams in Montana.

CALL REALDIUM TO FIND OUT IF THIS HAS BEEN DONE

Recent stability assessments have relied on previous geotechnical field investigations, laboratory analyses of materials and stability analysis models. The most recent inspection report (Billmayer & Hafferman, 2009) included a review of the 1992 seismic stability study by Harding Lawson. However, a critical review of updated seismic information was not apparently performed for the dam; the latest report stated agreement with the previous analyses performed in 1992.

REFERENCE

Verification of foundation conditions at the dam have not been performed, the original borings and test pits at the dam site were performed in 1971 and the most recent geotechnical borings in the vicinity of the dam were performed in 1991. Bedrock cores were not obtained and rock quality designations (RQDs) were not performed for the dam foundation. Additional stability analyses using recent state-of-the-art two-dimensional models have not been performed for the structure nor have finite element analyses of the dam structure stress conditions been performed.

Data regarding embankment movement over time has not been identified. There do not appear to be any surveyed settlement monuments on the dam crest; only visual assessments of embankment movement and erosion have been performed.

WHAT DID THESE ASSESSMENTS CONCLUDE?

Some discrepancies exist regarding previous hydrologic analyses performed for the Tailing Storage Facility (Schafer and Assoc., 1992) and recent hydrologic analyses of inflow design floods (Billmayer & Hafferman, 2009). The USGS regression equation methodology utilized in recent analyses likely does not have the accuracy required (67-79% std. error) for a structure such as the Tailing Storage Facility Dam at the Libby Mine. KDID

INTRODUCES A NEW NAME

Surface Mine Area

A few boreholes are reported to have been performed in the Surface Mine Area including some deep boreholes, although engineering-geologic data were not identified for the boreholes. One geologic log was identified for the monitoring well adjacent to the Glory Hole (MW-1; Well E) in the Surface Mine Area. It is not known what procedures were utilized in measurement of groundwater levels and what quality control procedures, if any, were utilized in the sampling and analysis of groundwater from the monitoring wells. Groundwater well sampling has been performed as part of the RI and various data gaps appear to exist for groundwater level data. The log of reported MW-2 (Well H) was not identified for review.

WE HAVE THE SAP I THINK

Insufficient geotechnical data exist in the Surface Mine Area to characterize site conditions with the objective of supporting evaluation of remedial alternatives in the FS. — WHAT'S THE BASIS OF THIS?

Data and information regarding reported underground mine workings and how such workings may affect the surface mine area, or other site areas, have not been identified.

Coarse Tailing Pile

Reference?

As mentioned above, no geotechnical engineering data were identified for the Coarse Tailing Pile other than anecdotal information regarding grain size of the coarse tailing materials. A geo-hydrologic report performed in the early 1980s (Zinner, 1982) presented general subsurface logs for areas east of the Coarse Tailing Pile, north of the surface mine. These indicated varying groundwater levels east of the Coarse Tailing Pile, but data was not identified to define groundwater levels within the Coarse Tailing Pile area. It is not known what quality control procedures were utilized in measurement of the groundwater levels or in characterization of subsurface materials.

The general quality and amount of data in the Coarse Tailing Pile Area, including surface and subsurface geotechnical and groundwater level data, are insufficient for analysis of FS alternatives.

Waste Rock Piles

Geotechnical data were not available for the waste rock piles and only one groundwater level data point was available at one of the waste rock piles. The Zinner report indicated artesian conditions between two waste rock piles (WRP-2 and WRP-3) but did not define the lateral extent of such conditions. Springs in the vicinity of the previous Zinner borings appear to confirm artesian conditions, although no additional monitoring well data is available in the vicinity. It is not known what quality control procedures were utilized in the collection of data prior to 2007.

The general quality and amount of data in the Waste Rock Piles Area, including surface and subsurface geotechnical and groundwater level data, are insufficient for analysis of FS alternatives.

Available groundwater level data for the site are presented in Table 5-1.

5.3 Data Quality Objectives

Data quality objectives (DQOs) define the type, quality, purpose and intended uses of data to be collected (EPA, 2006). The seven steps involved in the DQO process will be followed to provide an effective project plan and to provide sufficient information to support ~~key decisions regarding~~ remedial alternatives. The DQO process developed by EPA includes the following ~~seven~~ steps : 1) State the problem that the study is designed to address, 2) Identify the decisions to be made with the data obtained, 3) Identify the types of data inputs needed to make the decision, 4) Define the bounds (in space and time) of the study, 5) Define the decision rule which will be used to make decisions, 6) Define the acceptable limits on decision errors, and 7) Optimize the design using information identified in Steps 1-6.

An evaluation of Remedial Alternatives (including the No Action Alternative) in a FS

Statement of Problem

Remedial alternatives (including No Action) to be identified and evaluated in the FS require a sufficient amount of engineering information to support the evaluation of ~~implementability, effectiveness and cost~~. Various remaining questions need to be addressed for each of the areas, including the Tailing Storage Facility, the Coarse Tailing Pile, the Surface Mine Area and the Waste Rock Pile Area, to be evaluated in the FS.

Overall Protection of HH & Environment
Long Term Effectiveness & Permanence
Short Term Risk
Implementability Cost

Tailing Storage Facility:

Geotechnical data have been developed previously for the Tailing Storage Facility dam for stability and safety evaluations. Such data appear to be acceptable for defining the general safety of the dam along with regular inspections and maintenance procedures. However questions remain regarding the facility and additional data are needed to answer remaining questions for FS evaluations, including:

Overall Protection — State
Comply w/ ARARs — Community
Short Term Risk
Long Term Effectiveness & Permanence
Reduction of Tox, Vol, Mobility
Implementability
Cost

- What is the thickness of tailings within the impoundment? This is required to estimate the volume of tailings. This question can be answered by performing Cone Penetrometer Tests (CPT) within the impoundment.

- What is the consistency of the tailings at various depths within the impoundment? This is required to evaluate stability and liquefaction potential and can also be answered by the CPTs.

- What are the piezometric conditions within the impoundment upstream from the dam embankment? This is required is determine the stability, seepage conditions and liquefaction potential of the impoundment. This can be answered by the CPTs in combination with repair and measurement of the existing piezometer (P-O) within the impoundment area along with installation of a vibrating wire piezometer at this location.
  why?

- What is the current state of consistency (density, softness etc.) of dam embankment and foundation materials? This is required to update stability analyses and determine the current overall stability of the dam. This can be answered by a deep geotechnical boring through the maximum dam section into the underlying foundation bedrock along with associated sampling and geotechnical testing of various samples.

WOULD YOU EXPECT THE CONSISTENCY OF THESE MAT'LS TO CHANGE?

EMBANKMENT IS 135' HIGH

- What are the real-time piezometric variations in the dam embankment? This is required to better determine the potential rapid drawdown conditions within the embankment for stability analyses and the effects of varying piezometric conditions on the tailing impoundment and embankment. This can be answered by installing pressure transducers in the new boring and in at least two existing piezometers within the embankment with data loggers and possibly remote data transmittal.

- What are the verified foundation conditions for the tailing dam including the consistency of materials through the maximum dam section and the bedrock conditions beneath the dam? This is required for updated stability analyses of the tailing dam. This question can be answered by a deep borehole through the dam maximum section with sampling and geotechnical testing.

- What is a quantified amount of movement of the tailing dam over time? This is required to verify long-term stability in addition to visual assessments. This question can be answered by installation of a surface settlement monument on the dam crest.

– ADD:

- INFLOW ESTIMATES
- CAPACITY

Coarse Tailing Pile:

Various questions regarding the Coarse Tailing Pile remain for FS evaluations including:

- What is the thickness of Coarse Tailing Pile materials? This is required to estimate the volume of materials within the Coarse Tailing Pile and can be answered with a series of test pits throughout the facility excavated to native materials beneath the tailing materials.

- What are the characteristics and variability of materials within and over the Coarse Tailing Pile? This is required to determine the long-term erosion potential and stability of the facility. This can be answered by sampling of materials and testing for index geotechnical characteristics such as grain size analysis and Atterberg Limits.

- What is the groundwater level within or beneath the Coarse Tailing Pile? This is required to determine the piezometric conditions for assessment of long-term stability of the facility. This can be answered with installation of piezometers in two of the test pits with screened interval spanning the base of the tailing and original ground surface.

- What are the stability and conditions of the over-steepened area of the Coarse Tailing Pile? This is required to evaluate the long-term stability of the area. This may be answered by a complete initial geologic reconnaissance of the area based on standard protocol with an associated report, followed by surface settlement monuments or borehole inclinometers if determined to necessary.

Surface Mine Area:

Various questions remain regarding the Surface Mine Area including:

- What is the global stability of the Surface Mine Area? This is required to assess the long-term stability of the area, particularly the area with benching and side slopes, and can be answered by test pits with limited geotechnical testing of samples and by performing visual assessments of the benches and existing conditions in the steep portions of the area.

- What are the groundwater levels in the north and east portions of the Surface Mine Area? This is required to provide a better understanding of the overall potentiometric conditions throughout the mine area, and can be partially answered by restoration of Well J in the north part of the area.

- What is the volume of residential yard soils currently stored at the Surface Mine Area? This is required to determine the amount that may be used to place as cover over presently uncovered portions of the area. This can be answered by performing a review of the amount of soils hauled to the site or possibly a ground survey of the soil stockpile if necessary.

Questions regarding the reported underground mine workings may need to be addressed during the FS. However, no field explorations associated with this are recommended at this time.

REPORTED WHERE ?

Waste Rock Piles:

Various questions remain regarding the Waste Rock Pile Area including:

- What is the thickness of the waste rock piles? This is required to estimate the volume of the waste rock piles and can be answered utilizing data from boreholes developed through the wastes into the subsurface soils and the possible use of seismic refraction surveys on the waste rock piles.

- What is the composition of the waste materials, particularly the amount of decomposing vermiculite? This is required to determine the overall strength of the waste piles which is important to know to assess the long-term stability of the piles. This can be answered by obtaining samples of the wastes from borings and test pits and testing the materials for index geotechnical parameters and rock/soil types.

- What is the in-situ density of fine-grained materials in the waste rock piles? This is important to know to assess the long-term stability and creep potential of the waste rock piles and their impact on the surrounding land. This can be answered by analyzing moisture and density of relatively undisturbed samples of materials obtained from boreholes and test pits and comparing them with compaction test data on disturbed samples of waste rock materials.

- What is the amount of LA asbestos in the waste rock piles at various depths? This is required to determine the potential release of materials into the environment and also the stability of the piles. This can be answered by sampling materials from various depths in borings and test pits and testing for LA.

- What is the impact of high groundwater and potential artesian conditions on the waste rock piles, particularly those adjacent to previously reported high groundwater and artesian conditions (between WRP-2 and WRP-3; Zinner "Area 1")? This is required to assess the impact of potentially high piezometric conditions on the long-term stability of the waste rock piles, and to assess the potential for hydraulic release of LA materials to the environment from high groundwater conditions. This can be answered by installing boreholes with monitoring wells into WRP-2 and WRP-3 adjacent to the previously reported high groundwater and artesian conditions between these waste rock piles.

- What is the quantified movement of the waste piles over time? This is important to know to verify existing stability of the piles in addition to visual assessments, and can be partially answered by installation of surface settlement monuments at key locations on the waste piles and with borehole inclinometers.

Identify the Decision

WAY TOO GENERAL

The engineering data collected during the OU3 RI Phase III is intended to help EPA decide if and what remedial alternatives are feasible and necessary to protect human health and/or ecological receptors from unacceptable risks from asbestos and any other mining-related contaminants at the Tailing Storage Facility, Surface Mine Area, Coarse Tailing Pile, and Waste Rock Piles over the long term.

Identify Types of Data Needed

Engineering data needed for the various areas at OU3 include:

- Boring logs and test pits with associated logging in accordance with generally accepted ASTM standards and Cone Penetrometer Testing (CPT) in the tailing impoundment area;

- Subsurface soil sampling for bulk samples and relatively undisturbed samples;

- Geotechnical laboratory testing for index parameters such as grain size analysis and Atterberg Limits and strength/durability characteristics as necessary depending upon location of sampling;

- Installation of piezometers and groundwater monitoring wells for routine measurement and assessment of groundwater and phreatic surfaces through the various facilities;

- Geologic reconnaissance and field inspection of existing conditions is needed in some areas as a first step in evaluation of long-term stability;

- Installation of settlement monuments, or borehole inclinometers if determined to be necessary, at various locations to assess long-term embankment and waste pile/hillside stability concerns; and

- Survey data to determine the location and elevation of borings, test pits, monitoring wells, piezometers and settlement monuments or inclinometers and to verify existing slope conditions at the facilities.

- Seismic refraction survey information to define general subsurface conditions at the site.

Define Bounds of Study

The spatial bounds of the study include the total areas currently occupied by the Tailing Storage Facility, Coarse Tailing Pile, Surface Mine Area, and Waste Rock Piles at OU3.

The temporal bounds of the study will include one season of geotechnical sampling and monitoring new monitoring wells and settlement monuments, or borehole inclinometers, as applicable during a typical range of annual groundwater conditions.

Define the Decision Rule

The quality and results of engineering data from OU3 will not be used to determine if remedial action is necessary. However, used in combination with the decision rules for human and ecological risks and for potential environmental impacts, the data will be used to support FS evaluations.

Define Acceptable Limits on Decision Errors

Acceptable limits on decision errors for engineering data from OU3 will be based on established engineering principals, accepted ASTM standards and engineering judgment. Typically, if data are within reasonable limits for the type of material sampled and within the range of previous data for similar materials or previous data for the facilities, the data will be accepted.

Optimize the Design

The sampling design is based on the DQO process, the site characteristics and scale, and anticipated needs to support identification and evaluation of remedial alternatives in the FS process. Locations of investigation and sampling points may be varied somewhat in the field from the plan depending upon field conditions encountered.

5.4 Sampling Design

The sampling design includes various field geotechnical cone penetrometer tests, borings and test pits with associated logging, sampling and testing of soils, tailings and waste rock from the borings and test pits. The approximate location of the test pits and borings are shown on Figure 5-2 and the program is summarized in Table 5-2. Ranges of sample numbers are provided. The lower number indicates the minimum requirement. If the material is heterogeneous more samples than the minimum may be required based on field observation. Depending upon initial field investigations in various areas, additional geotechnical investigations may be necessary in addition to those indicated on Table 5-2. Such areas may include the potential diversion locations for Rainey Creek around the Tailing Storage Facility and the over-steepened area of the Coarse Tailing Pile.

Additional field geologic reconnaissance and inspection of existing conditions of various areas will also be performed as will land surveying of various features.

The location and elevation of all borings and test pits will be determined using survey-grade global positioning system (GPS) equipment. This equipment should provide the state plane coordinates to the nearest tenth of a foot and should provide the elevations to the nearest tenth of a foot based on feet above mean sea level.

All excavated test pits and boring cores will be documented with digital photography as necessary for each of the sampling locations.

Tailing Storage Facility

Previous investigations at the Tailing Storage Facility included a total of 10 borings developed for the 1992 geotechnical stability investigation of the impoundment dam. A total of 12 piezometers are annually monitored for dam safety inspections, none of which are within impounded tailings upstream of the dam. The original tailing dam design also included a series of borings in the vicinity of the dam which identify bedrock.

A total of three cone penetrometer tests (CPT) are proposed at the Tailing Storage Facility impoundment to verify the thickness and characteristics of the impounded tailing materials and subsurface conditions: at the upstream area (approximately 500 feet upstream of the embankment) where a levee was proposed in the 1992 report, one approximately 1,000 feet upstream from the dam and one approximately 2,000 feet upstream from the dam as shown on Figure 5-2. The location of these CPTs is approximate and may vary in the field depending upon accessibility.

NO - WE NEED TO SPECIFY EXACTLY WHAT WE WANT MEASURED.

Use of CPT methods should utilize low-ground-pressure equipment to access areas not possible with a conventional drill rig. This method does not extract samples of subsurface materials for laboratory testing, but rather utilizes electronic friction cone or piezocone equipment to record the penetration resistance of subsurface strata. This data presents a qualitative correlation to physical properties of materials present such as shear strength, bearing capacity, void ratios and pore pressures. Since data is continuously recorded, the depth, thickness and variation in the stratigraphy provide a complete profile of the materials encountered. The CPT data will be presented in standard format for each location with associated analyses of the data.

One deep geotechnical borehole should be drilled through the maximum tailing dam section at least 25 feet into underlying bedrock. This should be performed by a combination of auger rig and air rotary methods, as necessary, with sampling of embankment, tailing and bedrock

135'
25'
160' BORING

Size (Diameter) of Boring?

materials. It is estimated that the depth of this borehole will be approximately 175 to 180 feet. Disturbed samples of drill cutting will be collected as will split spoon samples in liners and undisturbed thin wall tube samples. Two thin-wall (Shelby) tube samples will be collected of fine-grained tailing and embankment materials. One or two rock core samples will be collected of the bedrock beneath the tailing dam. Index tests (grain size analyses and Atterberg Limits) will be performed on several samples and in-situ moisture-density tests will also be performed on tube samples. One Standard Proctor Compaction test will be performed on a bulk sample of embankment material for comparison with in-situ moisture-density tests. One undisturbed tube sample will be tested for triaxial shear. The borehole will be converted to a piezometer with screened interval in the lower portion of the embankment just above the foundation. A transducer will be installed in the piezometer, along with a data logger, to record real-time piezometric conditions within the dam embankment.

The existing non-functional piezometer in the impoundment area (P-0) should be repaired as necessary to assess the piezometric conditions in that area. It is recommended that a vibrating wire piezometer be installed to monitor pore pressure changes in the tailing materials. Such instruments provide a better assessment of piezometric conditions than open-tube piezometers in fine-grained materials such as tailings. Vibrating wire piezometers will be stainless steel units with durable pressure transducers capable of measuring pore pressures from -50 to 1,000 kilopascals (kPa; 145 pounds per square inch, psi) with an accuracy of plus or minus 0.1% full range. The unit shall be hermetically-sealed with durable cables and data loggers as necessary. The piezometer will be adequately protected with locking steel casings and concrete collars as necessary.

At least two of the existing piezometers in the dam embankment should be modified with installation of pressure transducers to measure real-time piezometric changes in the embankment. These should be installed in P-2 and PM-2 at a minimum, with possible installation in A-8. Data loggers should be installed to record all data with possible remote readout capability.

Specify

At least one concrete settlement monument will be placed on the tailing dam crest at the maximum section and will be surveyed to establish baseline data. This will provide needed quantification of embankment movements to complement and verify visual assessments and piezometer readings during periodic inspections. This will be a 10-inch diameter by 48-inch deep concrete cylinder installed vertically with the top approximately 3 inches above the existing ground surface. It may be either cast-in-place or precast concrete constructed with concrete having a minimum 28-day compressive strength of at least 3,500 pounds per square inch (psi). The top surface will have an embedded brass survey marker and will be surveyed for horizontal and vertical control from existing benchmarks; to the nearest 0.01 ft. Subsequent surveyed readings should then be performed twice per year through the FS period and following final remedial action. A survey point on the existing concrete principal spillway structure should also be established with associated baseline data.

Do you need Settlement Monument Here Also?

Visual assessments of existing ground conditions along potential diversion dam and diversion channel alignments for Rainey Creek upstream and adjacent to the Tailing Storage Facility will need to be made as a first step. If determined to be necessary during visual assessments, various test pits may be excavated at the potential diversion dam and channel locations with associated logging and sampling for index parameters.

Present this as option to CPT - specify data that would be collected

Geophysical survey procedures may be performed at the Tailing Storage Facility as necessary if access by CPT equipment is not possible in portions of the impoundment. This would be a ground seismic refraction survey.

Coarse Tailing Pile

Geotechnical investigations for the Coarse Tailing Pile will require four test pits. Approximate locations of the test pits are shown on Figure 5-2. The location of the test pits may vary in the field depending upon accessibility. – Give general guidelines

The test pits will be excavated with a large backhoe (track-hoe) to depths of approximately 10 to 12 feet. Slopes of test pits will be laid back to provide safe conditions as required by OSHA. The test pits will be logged by an experienced geologist or geotechnical engineer. Bulk samples of coarse tailing materials and underlying materials will be obtained and relatively undisturbed hand-driven samples will be obtained as possible. The hand-driven samples will be collected in 2-inch diameter by 4-inch long brass or stainless steel tubes. Alternatively 3-inch diameter by 6-inch long brass or stainless steel tubes could also be used.

Two test pits should be excavated near the toe of the Coarse Tailing Pile: one approximately 100 to 200 feet west of the pond and another approximately 800 to 1,000 feet west of this. These should be excavated to the base of the coarse tailing. Another test pit should be excavated about mid-way up the Coarse Tailing Pile slope in a relatively stable area and another should be excavated near the top of the Coarse Tailing Pile.

Bulk samples of cover soils, coarse tailing and subsurface materials should be collected from three of the test pits, as applicable. These samples should be tested for index properties including grain size analyses and Atterberg Limits as necessary depending amount of fines in the sample. In general, if the sample contains less than 10 percent fines (silt and clay passing the No. 200 sieve), Atterberg Limits will not be required, and the grain size analyses only need to be on the plus 200 sieve sizes. A few index property tests will be performed on bulk samples and in-situ moisture-density tests will be performed on relatively undisturbed tube samples. In addition, a few samples of existing cover soils should be tested for organic content. An assessment of the areal extent and thickness of existing cover soils will be made for the entire Coarse Tailing Pile Area.

Why?

Specify

Two of the test pits in the Coarse Tailing Pile area, CTP-TP1 and CTP-TP2, will have open-tube piezometers installed to determine groundwater levels and fluctuations beneath the area. These test pits will need to be excavated deep enough to reach groundwater level beneath the coarse tailing. The piezometers will consist of a 1.5-inch diameter PVC tube installed vertically with a screened interval from the base of the tailing to the groundwater level likely below the original ground surface. Granular fill filter material will be placed around the screened interval and compacted backfill and tailing placed around the remainder of the piezometer.

A geologic reconnaissance will be performed in the over-steepened area of the Coarse Tailing Pile as a first step. This reconnaissance should evaluate all surface conditions including visible surface features, seeps, if any, and evidence of movement with associated digital photographic documentation. A land survey should be performed of the over-steepened area including the adjoining land on both sides, above and below the area. If determined to be necessary following the initial investigations, settlement monuments will be installed at selected locations to monitor movement of the area over time. If movement of the over-steepened area is occurring, inclinometer(s) may be installed to further evaluate movements at depth.

Surface Mine Area

The Surface Mine Area will be investigated with test pits as shown on Figure 5-2 and with visual assessments of the area. Two test pits are recommended in the Surface Mine Area with associated logging and sampling of cover soils, mine wastes and subsurface materials. The thickness of cover soils should be recorded at each location and the soil horizon should be logged as necessary.

Bulk samples of subsurface materials should be obtained for index testing from at least one test pit: grain size analyses and Atterberg Limits Additionally, a sample of cover soil should be tested for organic content. An assessment of the areal extent and thickness of existing cover soils will be made in the Surface Mine Area.

WHY ?

A review of the amount of residential soils currently stockpiled on the Surface Mine Area should be made. The existing stockpile of residential soils may be surveyed if necessary to obtain an accurate volume of such materials.

Existing groundwater monitoring wells at the Site are being sampled as part of the RI. Data from this sampling will be used in the assessment of conditions in the Surface Mine Area and Waste Rock Pile Area. Existing Well J should be restored to obtain groundwater levels in that area, if possible.

CHECK AGAINST WHAT WE REQUIRED IN PHASE IIB

Data from existing monitoring wells Well E (MW-1), Well H (MW-2), Well D, Wells F and J (if possible), and previous well information from the Zinner Report, in addition to new monitoring wells to be installed will be utilized to gain a better understanding of the geo-hydrologic conditions in the Surface Mine-Waste Rock Pile-Carney Creek area. Conceptualization and characterization of the groundwater system in the project area will be performed in accordance with accepted standards.

Mine Waste Rock Piles

The three Waste Rock Piles will be investigated through a series of four test pits and three borings with two monitoring wells. The four test pits will include two on WRP-1 and one each on WRP-2 and WRP-3. Two or three of the test pits will be excavated near the top of the Waste Rock Piles and the remainder will be excavated in lower, accessible portions of the Waste Rock Piles.

One boring is proposed at the top of WRP-1 to assess the thickness of mine waste and subsurface soil horizon for stability. These borings should extend at least 5 feet into the native materials beneath the Waste Rock Pile for confirmation purposes. One boring each will be advanced through WRP-2 and WRP-3 within a few hundred feet of the previous borings which indicated artesian groundwater conditions. These should be located up-gradient and down-gradient of the previous boreholes performed in the Zinner Study Area 1. The exact locations will be field selected based on accessibility. Approximate locations of borings, monitoring wells and test pits shown on Figure 5-2 may vary in the field depending upon accessibility.

Two of the borings, in the WRP-2 and WRP-3 areas, will be developed as monitoring wells with 5 to 10 feet screened intervals within the groundwater zones encountered. It is anticipated that this will require 2-inch diameter Schedule 80 PVC casing. The MWs should be developed as necessary and monitored at least quarterly during the FS evaluation period. These monitoring wells should have protected steel pipe sections above ground surface with locking tops and concrete slabs at ground surface.

If possible, Well F should be rehabilitated to provide additional groundwater data between the surface mine area and the largest waste rock pile (WRP-1).

Three settlement monuments will be installed in the WRP areas to assess movement of these structures over time. These will be 10-inch diameter by 48-inch deep concrete cylinders installed vertically with the top approximately 3 inches above the existing ground surface. These may be either cast-in-place or precast concrete constructed with concrete having a minimum 28-day compressive strength of at least 3,500 pounds per square inch (psi). They will have brass survey markers embedded in the top and will be surveyed for horizontal and vertical control from existing benchmarks, to the nearest 0.01 ft.

One borehole inclinometer will be installed in WRP-B1. This inclinometer will allow an assessment of the overall movement of waste rock pile with depth. The inclinometer, along with the surface settlement monument located approximately 800 feet to the northwest, will provide an overall assessment of the movement of the largest waste rock pile over time.

Bulk samples of cover soils, waste rock and subsurface materials, as applicable should be obtained and tested for index parameters of grain size and Atterberg Limits, compaction and organic content of cover soils as necessary. Index tests will be performed on bulk samples, and a few organic content tests will be performed on surface soils and compaction tests will be performed on bulk, composite samples. The size of bulk samples may vary from large zip-lock plastic bags for index and organic content tests to 5-gallon bucket samples for compaction tests. An assessment will be made of the approximate volume of vermiculite in the Waste Piles based on visual assessments and sampling of borings and test pits.

Relatively undisturbed samples from borings or test pits will also be tested for in-situ moisture density. These in-situ moisture density tests will provide a definition of existing material conditions throughout the waste rock piles and some will be compared to the compaction tests to estimate the existing degree of compaction of materials. In addition, samples will be tested for strength to assess short and long-term stability of the Waste Rock Piles. The decomposition potential of materials within the waste rock piles will be evaluated through the use of freeze-thaw or slake-durability tests of selected samples of materials.

Geophysical survey methods may be utilized to determine subsurface conditions in areas between boreholes and in areas without any subsurface data. Such methods may consist of surface seismic refraction surveys or down-hole seismic surveys as applicable to the conditions.

Previous well information from the Zinner Report, in addition to new monitoring wells to be installed in boreholes (WRP-B2 and WRP-B3) will be utilized to gain a better understanding of the geo-hydrologic conditions in the Waste Rock Pile-Carney Creek area. Conceptualization and characterization of the groundwater system in the project area will be performed in accordance with accepted standards.

5.5 Analytical Requirements

The latest revision of the ASTM standards should be followed for all geotechnical soil and rock sampling and testing procedures. The following ASTM standards will be followed in sampling and analysis of geotechnical samples from OU3:

- Geotechnical Field Work should be performed in accordance with ASTM D-420 (Site Characterization for Engineering Design and Construction Purposes).

- Geologic reconnaissance procedures should be performed in accordance with standard ASTM procedures (Part 4.5 of ASTM D420-2003).

- Subsurface soils encountered in test pits and borings should be logged by an experienced geologist or geotechnical engineer in accordance with ASTM D-2487 (Classification of Soils for Engineering Purposes; Unified Soil Classification System) based on visual-manual procedures specified in ASTM D-2488 (Description and Identification of Soils; Visual-Manual Procedure).

- Standard penetration tests during boring shall be performed in accordance with ASTM D-1586 (Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils).

- Air rotary drilling will be performed in accordance with ASTM D-5782 (Standard Guide for Use of Direct Air-Rotary Drilling for Geoenvironmental Exploration and the Installation of Subsurface Water-Quality Monitoring Devices).

- Rock core drilling and sampling of rock beneath the tailing dam will be performed in accordance with ASTM D-2113 *(Standard Practice for Rock Core Drilling and Sampling of Rock for Site Investigations).*

- Downhole seismic testing will be performed in accordance with ASTM D-7400 (Standard Test Method for Downhole Seismic Testing).

- Seismic refraction investigations will be performed in accordance with ASTM D-5777 (Standard Guide for Using Seismic Refraction Method for Subsurface Investigations).

- Selection of geophysical subsurface investigation methods will be performed in accordance with ASTM D-6429 (Standard Guide for Selecting Surface Geophysical Methods).

- Cone penetrometer testing shall be performed in accordance with ASTM D-5778 (Standard Test Method for Performing Friction Cone and Piezocone Penetration Testing of Soils).

- Relatively undisturbed cohesive soil and tailings samples should be obtained using a Shelby Tube in accordance with ASTM D-1587 (Standard Practice for Thin-Walled Tube Geotechnical Sampling of Soils)

- Grain size analyses of soils should be performed in accordance with ASTM D-422 (Standard Test Method for Particle-Size Analysis of Soils) for sieve and hydrometer analyses.

- Atterberg Limits tests should be performed in accordance with ASTM D-4318 (Standard Test Methods for Liquid Limit, Plastic Limit and Plasticity Index of Soils).

- Rock core samples from beneath the tailing dam will be evaluated in accordance with ASTM D-5878 *(Standard Guide for Using Rock-Mass Classification Systems for Engineering Purposes)*.

- Relatively undisturbed samples should be tested for in-situ moisture and density in accordance with ASTM D-2216 (Standard Test Method for Laboratory Determination of Water [Moisture] Content of Soil and Rock by Mass) and ASTM D-2937 (Standard Test Method for Density of Soil in Place by the Drive-Cylinder Method).

- Standard compaction tests for waste materials should be performed in accordance with ASTM D-698 (Standard Test Method for Laboratory Compaction of Soil Using Standard Effort; Standard Proctor).

- Relative density of cohesionless granular materials, if any, should be tested in accordance with ASTM D-4253 (Standard Test Method for Maximum Index Density and Unit Weight of Soils Using a Vibratory Table) and ASTM D-4254 (Standard Test Method for Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density).

- Direct shear tests of undisturbed and remolded soils should be performed in accordance with ASTM D-3080 (Standard Test Method for Direct Shear Test of Soils under Consolidated Drained Conditions).

- Slake-Durability tests, if performed on materials obtained from the waste rock piles, should be performed in accordance with ASTM D-5312 (Standard Test Method for Slake Durability of Shales and Similar Weak Rocks).

- Freeze-Thaw tests, if performed on materials obtained from the waste rock piles, should be performed in accordance with ASTM D-4644 (Standard Test Method for Evaluation of Durability of Rock for Erosion Control under Freeze-Thaw Conditions).

- Organic content of soils should be performed in accordance with ASTM D-2974 (Standard Test Methods for Moisture, Ash, and Organic Matter of Peat and Other Organic Soils).

- Monitoring wells will be installed in accordance with ASTM 5092 (Design and Installation of Ground Water Monitoring Wells in Aquifers).

- Vibrating wire piezometers will be installed in accordance with USBR or U.S. Army Corps of Engineers requirements.

- Borehole inclinometers will be installed and monitored in accordance with ASTM D-6230 (Test Method for Monitoring Ground Movement Using Probe-Type Inclinometers).

- Monitoring wells will be protected in accordance with ASTM D-5787 (Standard Practice for Monitoring Well Protection).

- Groundwater conditions in the Surface Mine and Waste Rock Pile Areas should be evaluated in accordance with ASTM D-5979 (Standard Guide for Conceptualization and Characterization of Ground-Water Systems).

5.6 Quality Control

Quality control will be performed on a continuous basis by site personnel as work progress in the field. Field record books will be maintained as necessary and field logs will be maintained and copied daily to eliminate the possibility of lost data. Approximately 5 to 10 percent additional samples will be collected in the field, beyond those specified, for later testing if test results appear to be in error.

Samples will be handled, packaged, labeled and shipped to the testing laboratory in accordance with accepted ASTM and EPA standards. All testing by the laboratory will be performed in accordance with accepted ASTM standards including all required data and information reporting required by the standards.

Field logs of borings and test pits will be reviewed and corrected as necessary based on the laboratory testing. The geotechnical report will be developed by consultants for W.R. Grace and reviewed by the various parties involved in the program.

Surveying for location and elevation of borings and test pits will be performed in accordance with accepted survey standards of the American Congress on Surveying and Mapping (ACSM) and the National Society of Professional Surveyor (NSPS).

Table 5-1: Groundwater Level Data

Description	Location	Sample Date	Water Level Elevation (ft)	Water Level (ft bgs)	Notes
Kootenai Development Impoundment Dam Piezometers (PVC Open-Tube, except P-O)	A8	2/20/2009	2797.8	8.2	
		1/15/2009	2797.7	8.3	
		12/1/2008	2797.8	8.2	
		10/30/2008	2797.8	8.2	
		10/2/2008	2797.9	8.1	
		8/8/2008	2799.0	7.0	
		7/3/2008	2801.3	4.6	
		6/3/2008	2803.0	2.9	
		5/20/2008	2803.3	2.7	
		5/16/2008	2802.1	3.9	
		4/23/2008	2798.4	7.6	
		3/10/2008	2797.6	8.4	
		5/8/2007	2800.7	5.2	
	P-0	-	-	-	Not Functional
	P1	5/8/2007	-	-	Dry
	P2	2/20/2009	2722.3	119.9	
		1/15/2009	2721.8	120.4	
		12/1/2008	2721.6	120.6	
		10/30/2008	2723.1	119.2	
		10/2/2008	2724.3	117.9	
		8/8/2008	2726.5	115.8	
		7/3/2008	2736.8	105.4	
		6/3/2008	2754.7	87.5	
		5/20/2008	2751.8	90.5	
		5/16/2008	2750.9	91.3	
		4/23/2008	2727.8	114.4	
		3/10/2008	2722.6	119.7	
		2/7/2008	2722.1	120.1	
		5/8/2007	2734.6	107.6	
	P3	5/8/2007	-	-	Dry
	P4	5/8/2007	2746.2	105.2	
	P5	5/8/2007	2763.8	103.6	
	PM1	2/20/2009	2757.6	53.7	
		1/15/2009	2757.4	53.9	
		12/1/2008	2757.4	53.9	
		10/30/2008	2757.4	53.9	
		10/2/2008	2757.4	53.9	
		8/8/2008	2758.2	53.1	
		7/3/2008	2761.6	49.7	
		6/3/2008	2762.9	48.4	
		5/20/2008	2763.1	48.2	
		5/16/2008	2764.9	46.5	
		4/23/2008	2761.1	50.2	
		3/10/2008	2759.8	51.5	
		5/8/2007	2761.7	49.6	

Table 5-1 Continued

Description	Location	Sample Date	Water Level Elevation (ft)	Water Level (ft bgs)	Notes
Kootenai Development Impoundment Dam Piezometers (PVC) Continued	PM2	2/20/2009	2734.0	103.7	
		1/15/2009	2733.7	104.1	
		12/1/2008	2733.7	104.1	
		10/30/2008	2733.9	103.9	
		8/8/2008	2736.7	101.1	
		7/3/2008	2740.3	97.5	
		6/3/2008	2747.1	90.7	
		5/20/2008	2749.8	88.0	
		5/16/2008	2749.4	88.4	
		4/23/2008	2736.7	101.1	
		3/10/2008	2734.3	103.5	
		5/8/2007	2741.6	96.2	
	PM3	5/8/2007	2767.5	51.6	
	PM4	5/8/2007	-	-	Dry
	PM5	5/8/2007	-	-	Dry
	PM6	5/8/2007	-	-	Dry
"CCC Well" in Carney Creek drainage, upstream of pond below fine tailings	Well A	7/22/2008	3349.6	1.8	
		9/29/2008	3351.0	0.4	
		10/1/2007	3348.1	3.3	
In clearing across small creek south of tailings dam, upstream of Watergate	Well C	7/22/2008	2764.6	22.8	
		9/29/2008	2763.3	24.1	
		10/1/2007	2763.4	24.1	
In pump house above (east of) tailings pond dam, potable supply well. Well log dated 11/28	Well D	7/23/2008	3583.6	241.5	
		9/30/2008	3581.2	243.9	
		10/1/2007	3579.6	245.5	
		2/25/1986	3584.2	240.9	
"MW-1" just off road on broad top level, ESE of pump house	Well E	7/23/2008	3789.0	172.6	
		9/30/2008	3770.6	191.1	
		10/1/2007	3883.4	78.3	
		9/22/2000	3771.2	190.5	
2-inch PVC well on edge of slope above (north) of Carney Cr.	Well F	10/1/2007	3406.4	213.9	Poor Condition
West of Mine "MW-2"	Well H	7/24/2008	3281.3	59.9	
		9/30/2008	-	-	Dry
		10/4/2000	3287.4	53.8	
Zinner Well Study Area 1	Z26	7/1/1981	-	-	no wl
	Z27	7/1/1981	3758.0	-2.0	Artesian
	Z28	7/1/1981	3755.0	-12.5	Artesian
	Z29	7/1/1981	-	-	no wl
	Z30	7/1/1981	3751.0	-9.7	Artesian
	Z31	7/1/1981	3748.0	-2.0	Artesian
	Z32	7/1/1981	3744.0	8.6	Artesian
	Z33	7/1/1981	3741.0	4.6	Artesian
	Z34	7/1/1981	3740.0	1.5	Artesian
	Z35	7/1/1981	3734.0	14.7	

Table 5-1 Continued

Description	Location	Sample Date	Water Level Elevation (ft)	Water Level (ft bgs)	Notes
Zinner Well Study Area 1 Continued	Z36	7/1/1981	3735.0	17.6	
	Z37	7/1/1981	3666.0	80.6	
	Z38	7/1/1981	-	-	no wl
	Z39	7/1/1981	-	-	no wl
	Z44	7/1/1981	3615.0	95.3	
	Z45	7/1/1981	-	-	no wl
	Z46	7/1/1981	3714.0	-4.7	Artesian
	Z47	7/1/1981	3713.0	-9.2	Artesian
	Z48	7/1/1981	3705.0	-16.6	
	Z49	7/1/1981	3710.0	-10.2	Artesian
	Z50	7/1/1981	3702.0	-1.5	
	Z51	7/1/1981	3696.0	5.7	
	Z52	7/1/1981	3649.0	56.9	
	Z53	7/1/1981	3627.0	84.8	
	Z54	7/1/1981	3550.0	158.2	
	Z55	7/1/1981	3545.0	161.2	
	Z56	7/1/1981	-	-	no wl
	Z57	7/1/1981	3552.0	139.4	
	Z58	7/1/1981	-	-	
	Z59	7/1/1981	-	-	no wl
	Z60	7/1/1981	3516.0	167.2	
	Z62	7/1/1981	-	-	no wl
	Z63	7/1/1981	3544.0	143.9	
	Z64	7/1/1981	3618.0	68.3	
	Z65	7/1/1981	-	-	no wl
	Z66	7/1/1981	3675.0	2.8	
	Z67	7/1/1981	-	-	
	Z68	7/1/1981	3664.0	70.9	
	Z69	7/1/1981	3712.0	12.9	
	Z70	7/1/1981	3718.0	1.6	Artesian
	Z71	7/1/1981	3732.0	-20.3	Artesian
	Z72	7/1/1981	3728.0	-2.7	
Zinner Well Study Area 2	Z83	7/1/1981	-	-	no static wl
	Z84	7/1/1981	3417.0	63.2	
	Z85	7/1/1981	3414.0	70.0	
	Z86	7/1/1981	3348.0	122.2	
	Z87	7/1/1981	3334.0	123.7	
	Z88	7/1/1981	3338.0	123.0	
	Z89	7/1/1981	3351.0	89.0	
	Z90	7/1/1981	3445.0	17.7	
	Z91	7/1/1981	-	-	no static wl
	Z92	7/1/1981	3445.0	82.3	
	Z93	7/1/1981	3446.0	78.2	
	Z94	7/1/1981	3456.0	74.9	
	Z95	7/1/1981	3479.0	49.0	

Table 5-1 Continued

Description	Location	Sample Date	Water Level Elevation (ft)	Water Level (ft bgs)	Notes
Zinner Well Study Area 2 Continued	Z96	7/1/1981	3485.0	41.3	
	Z97	7/1/1981	-	-	no static wl
	Z98	7/1/1981	-	-	no static wl
	Z99	7/1/1981	3467.0	76.4	
Zinner Well Study Area 3	Z73	7/1/1981	3418.0	72.6	
	Z74	7/1/1981	3407.0	85.0	
	Z75	7/1/1981	3410.0	81.7	
	Z76	7/1/1981	3443.0	46.4	
	Z78	7/1/1981	-	-	no static wl
	Z79	7/1/1981	3466.0	93.1	
	Z80	7/1/1981	-	-	no static wl
	Z81	7/1/1981	-	-	no static wl
	Z82	7/1/1981	-	-	no static wl
	Z100	7/1/1981	-	-	no static wl
	Z101	7/1/1981	3506.0	145.9	
	Z102	7/1/1981	-	-	no static wl
	Z103	7/1/1981	3529.0	111.7	
	Z104	7/1/1981	-	-	no static wl
	Z105	7/1/1981	3531.0	91.1	
	Z106	7/1/1981	-	-	no static wl
	Z107	7/1/1981	3540.0	99.0	
	Z108	7/1/1981	-	-	no static wl
	Z109	7/1/1981	3544.0	113.4	
	Z110	7/1/1981	-	-	no static wl
	Z111	7/1/1981	3556.0	100.5	
	Z112	7/1/1981	3562.0	90.1	
	Z113	7/1/1981	3608.0	44.0	
	Z116	7/1/1981	3418.0	96.4	
	Z117	7/1/1981	3458.0	62.2	
	Z118	7/1/1981	3410.0	111.3	
	Z119	7/1/1981	3458.0	69.4	
	Z120	7/1/1981	3460.0	69.5	
	Z121	7/1/1981	3460.0	67.1	
	Z122	7/1/1981	-	-	no static wl
	Z123	7/1/1981	-	-	no static wl
	Z124	7/1/1981	-	-	no static wl
Carney Creek Seeps/Springs	CCS-1	6/28/2008	3472.5	0.0	Seep
	CCS-11	6/28/2008	3723.5	0.0	Spring
	CCS-14	6/28/2008	3761.1	0.0	Spring
	CCS-16	6/28/2008	3676.3	0.0	Seep
	CCS-6	6/28/2008	3285.2	0.0	Seep
	CCS-8	6/28/2008	3254.1	0.0	Seep
	CCS-9	6/28/2008	3005.7	0.0	Seep

Table 5-2: Summary of Geotechnical Investigations

Boring, Test Pit or Item ID	*Bulk Samples*	*Undisturbed* Samples	*Index* Tests	*Moisture-* Density Tests	*Compaction* Tests	*Strength* Tests	*Rock Durability* Tests	*Organic* Content Tests	*Piezometers or* Monitoring Well	*Comment*
TSF-B1	2-3	2	3-4	2-3	1	1 TX	1-2 RQDs		Install New Piezo. & Transducer & Data Logger	At Max. Dam Section
TSF CPT-1 to 3	-----	-------	------	------	------	-------	-------	------	------	Std. CPT Rpt
TSF Existing Piezo. P-0	------	------	------	------	-------	------	-------	------	Install VW Piezo.	Repair Piezo.;Add Data Logger
Existing P-2 and PM-2									Install Transducers	Data Loggers
CTP-TP1	1-2	1	1-2	1	--------	------	-------	1	Install New Piezo.	----
CTP-TP2	1-2	1	1-2	1	-------	------	-----	1	Install New Piezo.	----
CTP-TP4	1-2	1	1	1	------	------	------	1	-------	-----
CTP Geologic Recon.	------	------	------	------	------	-----	------	-----	-------	Possible SM/Inclin
SMA-TP2	1	-	1				-------	1	-------	------
WRP-B1	2-3	2-3	2-3	1-2	1	1-TX	-------	------	--------	Install Inclinometer
WRP-B2	1-2	1-2	1-2	1-2	-----	------	------	------	-------	New MW
WRP-B3	1-2	1-2	1-2	1-2	------	-------	--------		--------	New MW
WRP-TP1	1-2	1	1	1	-------	------	1 F-T or S-D	1	-------	---
WRP-TP3	1-2	1	1	1	-----	------		1		----
WRP-TP4	1-2	1	1	1	1	1-DS	1 S-D or F-T	-----	------	----

Notes:

TSF denotes Tailing Storage Facility
CPT denotes Cone Penetrometer Test
F-T denotes Freeze-Thaw Test
CTP denotes Coarse Tailing Pile
WRP denotes Waste Rock Pile
SMA denotes Surface Mine Area
DS denotes Direct Shear Test.
VW denotes Vibrating Wire Piezometers
S-D denotes Slake-Durability Test
TX denotes Triaxial Shear Test

Settlement Monuments at TSF and WRP areas not shown and existing MWs not indicated although water level measurements required from all existing MWs

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ATTACHMENT 5

rough draft geotech work
Bonita Lavelle to: Robert.R.Marriam 12/16/2010 01:09 PM

Dear Bob,

Attached is a very rough draft of the sampling program for the geotech investigation. Let's discuss when you have a chance.

Bonnie

geotech sampling design.docx

table 5-2.docx

Figure 5-2-22July2010.pdf

5.4 Sampling Design

The sampling design includes various field geotechnical cone penetrometer tests, borings and test pits with associated logging, sampling and testing of soils, tailings and waste rock from the borings and test pits. The approximate location of the test pits and borings are shown on Figure 5-2 and the program is summarized in Table 5-2. Ranges of sample numbers are provided. The lower number indicates the minimum number of samples that are required. If the material is heterogeneous, more samples than the minimum may be required to ensure sampling is representative. The decision to collect more samples will be made in the field based on field observation. Depending upon initial field investigations in various areas, additional geotechnical investigations may be necessary in addition to those indicated on Table 5-2 (e.g., at the over-steepened area of the Coarse Tailing Pile).

Additional field geologic reconnaissance and inspection of existing conditions of various areas will also be performed as will land surveying of various features.

The location and elevation of all borings and test pits will be determined using survey-grade global positioning system (GPS) equipment. This equipment should provide the state plane coordinates to the nearest tenth of a foot and should provide the elevations to the nearest tenth of a foot based on feet above mean sea level.

All excavated test pits and boring cores will be documented with digital photography as necessary for each of the sampling locations.

Tailing Storage Facility

Data needed to evaluate the stability and liquefaction potential of the tailings impounded in the Tailings Storage Facility:

A total of three cone penetrometer tests (CPT) are needed at the Tailing Storage Facility impoundment to determine the thickness (to support volume calculations) and characteristics of the impounded tailing materials and subsurface conditions: at the upstream area (approximately 500 feet upstream of the embankment) where a levee was proposed in the 1992 report, one approximately 1,000 feet upstream from the dam and one approximately 2,000 feet upstream from the dam as shown on Figure 5-2. The location of these CPTs is approximate and may vary in the field depending upon accessibility.

CPT methods should utilize low-ground-pressure equipment to access areas not possible with a conventional drill rig. This method does not extract samples of subsurface materials for laboratory testing, but rather utilizes electronic friction cone or piezocone equipment to record the penetration resistance of subsurface strata. This data presents a qualitative correlation to physical properties of materials present such as shear strength, bearing capacity, void ratios and pore pressures. Since data is continuously recorded, the depth, thickness and variation in the stratigraphy provide a complete profile of the materials encountered. The CPT data will be presented in standard format for each location with associated analyses of the data.

Data needed to perform an updated stability analysis of the impoundment dam:

One deep geotechnical borehole should be drilled through the maximum tailing dam section at least 25 feet into underlying bedrock to collect information to allow a determination of the current state of consistency (density, softness, etc.) of the dam embankment and foundation materials. The drilling of the geotechnical borehole should be performed by a combination of auger rig and air rotary methods, as necessary, with sampling of embankment, tailing and bedrock materials. It is estimated that the depth of this borehole will be approximately 175 to 180 feet. Disturbed samples of drill cutting will be collected as will split spoon samples in liners and undisturbed thin wall tube samples. Two thin-wall (Shelby) tube samples will be collected of fine-grained tailing and embankment materials. One or two rock core samples will be collected of the bedrock beneath the tailing dam. Index tests (grain size analyses and Atterberg Limits) will be performed on several samples and in-situ moisture-density tests will also be performed on tube samples. One Standard Proctor Compaction test will be performed on a bulk sample of embankment material for comparison with in-situ moisture-density tests. One undisturbed tube sample will be tested for triaxial shear. The borehole will be converted to a piezometer with screened interval in the lower portion of the embankment just above the foundation. A transducer will be installed in the piezometer, along with a data logger, to record real-time piezometric conditions within the dam embankment.

The existing non-functional piezometer in the impoundment area (P-0) should be repaired as necessary to assess the piezometric conditions in that area. It is recommended that a vibrating wire piezometer be installed to monitor pore pressure changes in the tailing materials. Such instruments provide a better assessment of piezometric conditions than open-tube piezometers in fine-grained materials such as tailings. Vibrating wire piezometers will be stainless steel units with durable pressure transducers capable of measuring pore pressures from -50 to 1,000 kilopascals (kPa; 145 pounds per square inch, psi) with an accuracy of plus or minus 0.1% full range. The unit shall be hermetically-sealed with durable cables and data loggers as necessary. The piezometer will be adequately protected with locking steel casings and concrete collars as necessary.

At least two of the existing piezometers in the dam embankment should be modified with installation of pressure transducers to measure real-time piezometric changes in the embankment. These should be installed in P-2 and PM-2 at a minimum, with possible installation in A-8. Data loggers should be installed to record all data with possible remote readout capability.

The piezometer data will be used to determine the potential rapid drawdown conditions within the embankment to support an updated stability analysis. The data will also be used to determine the effects of varying piezometric conditions on the tailings impoundment and embankment.

At least one concrete settlement monument will be placed on the tailing dam crest at the maximum section and will be surveyed to establish baseline data. This will provide the data needed to quantify embankment movements. The data will complement and verify visual assessments and piezometer readings taken during periodic inspections. This will be a 10-inch diameter by 48-inch deep concrete cylinder installed vertically with the top approximately 3 inches above the existing ground surface. It may be either cast-in-place or precast concrete constructed with concrete having a minimum 28-day compressive strength of at least 3,500 pounds per square inch (psi). The top surface will have an embedded brass survey marker and will be surveyed for horizontal and vertical control from existing benchmarks; to the nearest 0.01 ft. Subsequent surveyed readings should then be performed twice per year through the FS period and following final remedial action. A survey point on the existing concrete principal spillway structure should also be established with associated baseline data.

Geophysical survey procedures may be performed at the Tailing Storage Facility as necessary if access by CPT equipment is not possible in portions of the impoundment. This would be a ground seismic refraction survey.

Coarse Tailing Pile

Data needed to determine the long-term stability and erosion potential of the coarse tailings pile:

Geotechnical investigations for the Coarse Tailing Pile will require four test pits. Approximate locations of the test pits are shown on Figure 5-2. The location of the test pits may vary in the field depending upon accessibility.

The test pits will be excavated with a large backhoe (track-hoe) to depths of approximately 10 to 12 feet. Slopes of test pits will be laid back to provide safe conditions as required by OSHA. The test pits will be logged by an experienced geologist or geotechnical engineer. Bulk samples of coarse tailing materials and underlying materials will be obtained and relatively undisturbed hand-driven samples will be obtained as possible. The hand-driven samples will be collected in 2-inch diameter by 4-inch long brass or stainless steel tubes. Alternatively 3-inch diameter by 6-inch long brass or stainless steel tubes could also be used.

Two test pits should be excavated near the toe of the Coarse Tailing Pile: one approximately 100 to 200 feet west of the pond and another approximately 800 to 1,000 feet west of this. These should be excavated to the base of the coarse tailing. Another test pit should be excavated about mid-way up the Coarse Tailing Pile slope in a relatively stable area and another should be excavated near the top of the Coarse Tailing Pile.

Bulk samples of cover soils, coarse tailing and subsurface materials should be collected from three of the test pits, as applicable. These samples should be tested for index properties including grain size analyses and Atterberg Limits as necessary depending amount of fines in the sample. In general, if the sample contains less than 10 percent fines (silt and clay passing the No. 200 sieve), Atterberg Limits will not be required, and the grain size analyses only need to be on the plus 200 sieve sizes. A few index property tests will be performed on bulk samples and in-situ moisture-density tests will be performed on relatively undisturbed tube samples. In addition, a few samples of existing cover soils should be tested for organic content. An assessment of the areal extent and thickness of existing cover soils will be made for the entire Coarse Tailing Pile Area.

Two of the test pits in the Coarse Tailing Pile area, CTP-TP1 and CTP-TP2, will have open-tube piezometers installed to determine groundwater levels and fluctuations beneath the area. These test pits will need to be excavated deep enough to reach groundwater level beneath the coarse tailing. The piezometers will consist of a 1.5-inch diameter PVC tube installed vertically with a screened interval from the base of the tailing to the groundwater level likely below the original ground surface. Granular fill filter material will be placed around the screened interval and compacted backfill and tailing placed around the remainder of the piezometer.

A geologic reconnaissance will be performed in the over-steepened area of the Coarse Tailing Pile as a first step. This reconnaissance should evaluate all surface conditions including visible surface features, seeps, if any, and evidence of movement with associated digital photographic documentation. A land survey should be performed of the over-steepened area including the adjoining land on both sides, above and below the area. If determined to be necessary following the initial investigations, settlement monuments will be installed at selected locations to monitor movement of the area over time. If movement of the over-steepened area is occurring, inclinometer(s) may be installed to further evaluate movements at depth.

Surface Mine Area

The Surface Mine Area will be investigated with test pits as shown on Figure 5-2 and with visual assessments of the area. Two test pits are recommended in the Surface Mine Area with associated logging and sampling of cover soils, mine wastes and subsurface materials. The thickness of cover soils should be recorded at each location and the soil horizon should be logged as necessary.

Bulk samples of subsurface materials should be obtained for index testing from at least one test pit: grain size analyses and Atterberg Limits. Additionally, a sample of cover soil should be tested for organic content. An assessment of the areal extent and thickness of existing cover soils will be made in the Surface Mine Area.

A review of the amount of residential soils currently stockpiled on the Surface Mine Area should be made. The existing stockpile of residential soils may be surveyed if necessary to obtain an accurate volume of such materials.

Existing groundwater monitoring wells at the Site are being sampled as part of the RI. Data from this sampling will be used in the assessment of conditions in the Surface Mine Area and Waste Rock Pile Area. Existing Well J should be restored to obtain groundwater levels in that area, if possible.

Data from existing monitoring wells Well E (MW-1), Well H (MW-2), Well D, Wells F and J (if possible), and previous well information from the Zinner Report, in addition to new monitoring wells to be installed will be utilized to gain a better understanding of the geo-hydrologic conditions in the Surface Mine-Waste Rock Pile-Carney Creek area. Conceptualization and characterization of the groundwater system in the project area will be performed in accordance with accepted standards.

Mine Waste Rock Piles

The three Waste Rock Piles will be investigated by installing four test pits and three borings with two monitoring wells. The four test pits will include two on WRP-1 and one each on WRP-2 and WRP-3. Two or three of the test pits will be excavated near the top of the Waste Rock Piles and the remainder will be excavated in lower, accessible portions of the Waste Rock Piles.

One boring is proposed at the top of WRP-1 to assess the thickness of mine waste and subsurface soil horizon for stability. These borings should extend at least 5 feet into the native materials beneath the Waste Rock Pile for confirmation purposes. One boring each will be advanced through WRP-2 and WRP-3 within a few hundred feet of the previous borings which indicated artesian groundwater conditions. These should be located up-gradient and down-gradient of the previous boreholes performed in the Zinner Study Area 1. The exact locations will be field selected based on accessibility. Approximate locations of borings, monitoring wells and test pits shown on Figure 5-2 may vary in the field depending upon accessibility.

Two of the borings, in the WRP-2 and WRP-3 areas, will be developed as monitoring wells with 5 to 10 feet screened intervals within the groundwater zones encountered. It is anticipated that this will require 2-inch diameter Schedule 80 PVC casing. The MWs should be developed as necessary and monitored at least quarterly during the FS evaluation period. These monitoring wells should have protected steel pipe sections above ground surface with locking tops and concrete slabs at ground surface.

If possible, Well F should be rehabilitated to provide additional groundwater data between the surface mine area and the largest waste rock pile (WRP-1).

Three settlement monuments will be installed in the WRP areas to assess movement of these structures over time. These will be 10-inch diameter by 48-inch deep concrete cylinders installed vertically with the top approximately 3 inches above the existing ground surface. These may be either cast-in-place or precast concrete constructed with concrete having a minimum 28-day compressive strength of at least 3,500 pounds per square inch (psi). They will have brass survey markers embedded in the top and will be surveyed for horizontal and vertical control from existing benchmarks, to the nearest 0.01 ft.

One borehole inclinometer will be installed in WRP-B1. This inclinometer will allow an assessment of the overall movement of waste rock pile with depth. The inclinometer, along with the surface settlement monument located approximately 800 feet to the northwest, will provide an overall assessment of the movement of the largest waste rock pile over time.

Bulk samples of cover soils, waste rock and subsurface materials, as applicable should be obtained and tested for index parameters of grain size and Atterberg Limits, compaction and organic content of cover soils as necessary. Index tests will be performed on bulk samples, and a few organic content tests will be performed on surface soils and compaction tests will be performed on bulk, composite samples. The size of bulk samples may vary from large zip-lock plastic bags for index and organic content tests to 5-gallon bucket samples for compaction tests. An assessment will be made of the approximate volume of vermiculite in the Waste Piles based on visual assessments and sampling of borings and test pits.

Relatively undisturbed samples from borings or test pits will also be tested for in-situ moisture density. These in-situ moisture density tests will provide a definition of existing material conditions throughout the waste rock piles and some will be compared to the compaction tests to estimate the existing degree of compaction of materials. In addition, samples will be tested for strength to assess short and long-term stability of the Waste Rock Piles. The decomposition potential of materials within the waste rock piles will be evaluated through the use of freeze-thaw or slake-durability tests of selected samples of materials.

Geophysical survey methods may be utilized to determine subsurface conditions in areas between boreholes and in areas without any subsurface data. Such methods may consist of surface seismic refraction surveys or down-hole seismic surveys as applicable to the conditions.

Previous well information from the Zinner Report, in addition to new monitoring wells to be installed in boreholes (WRP-B2 and WRP-B3) will be utilized to gain a better understanding of the geo-hydrologic conditions in the Waste Rock Pile-Carney Creek area. Conceptualization and characterization of the groundwater system in the project area will be performed in accordance with accepted standards.

5.5 Analytical Requirements

The latest revision of the ASTM standards should be followed for all geotechnical soil and rock sampling and testing procedures. The following ASTM standards will be followed in sampling and analysis of geotechnical samples from OU3:

- Geotechnical Field Work should be performed in accordance with ASTM D-420 (Site Characterization for Engineering Design and Construction Purposes).

- Geologic reconnaissance procedures should be performed in accordance with standard ASTM procedures (Part 4.5 of ASTM D420-2003).

- Subsurface soils encountered in test pits and borings should be logged by an experienced geologist or geotechnical engineer in accordance with ASTM D-2487 (Classification of Soils for Engineering Purposes; Unified Soil Classification System) based on visual-manual procedures specified in ASTM D-2488 (Description and Identification of Soils; Visual-Manual Procedure).

- Standard penetration tests during boring shall be performed in accordance with ASTM D-1586 (Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils).

- Air rotary drilling will be performed in accordance with ASTM D-5782 (Standard Guide for Use of Direct Air-Rotary Drilling for Geoenvironmental Exploration and the Installation of Subsurface Water-Quality Monitoring Devices).

- Rock core drilling and sampling of rock beneath the tailing dam will be performed in accordance with ASTM D-2113 *(Standard Practice for Rock Core Drilling and Sampling of Rock for Site Investigations).*

- Downhole seismic testing will be performed in accordance with ASTM D-7400 (Standard Test Method for Downhole Seismic Testing).

- Seismic refraction investigations will be performed in accordance with ASTM D-5777 (Standard Guide for Using Seismic Refraction Method for Subsurface Investigations).

- Selection of geophysical subsurface investigation methods will be performed in accordance with ASTM D-6429 (Standard Guide for Selecting Surface Geophysical Methods).

- Cone penetrometer testing shall be performed in accordance with ASTM D-5778 (Standard Test Method for Performing Friction Cone and Piezocone Penetration Testing of Soils).

- Relatively undisturbed cohesive soil and tailings samples should be obtained using a Shelby Tube in accordance with ASTM D-1587 (Standard Practice for Thin-Walled Tube Geotechnical Sampling of Soils)

- Grain size analyses of soils should be performed in accordance with ASTM D-422 (Standard Test Method for Particle-Size Analysis of Soils) for sieve and hydrometer analyses.

- Atterberg Limits tests should be performed in accordance with ASTM D-4318 (Standard Test Methods for Liquid Limit, Plastic Limit and Plasticity Index of Soils).

- Rock core samples from beneath the tailing dam will be evaluated in accordance with ASTM D-5878 *(Standard Guide for Using Rock-Mass Classification Systems for Engineering Purposes)*.

- Relatively undisturbed samples should be tested for in-situ moisture and density in accordance with ASTM D-2216 (Standard Test Method for Laboratory Determination of Water [Moisture] Content of Soil and Rock by Mass) and ASTM D-2937 (Standard Test Method for Density of Soil in Place by the Drive-Cylinder Method).

- Standard compaction tests for waste materials should be performed in accordance with ASTM D-698 (Standard Test Method for Laboratory Compaction of Soil Using Standard Effort; Standard Proctor).

- Relative density of cohesionless granular materials, if any, should be tested in accordance with ASTM D-4253 (Standard Test Method for Maximum Index Density and Unit Weight of Soils Using a Vibratory Table) and ASTM D-4254 (Standard Test Method for Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density).

- Direct shear tests of undisturbed and remolded soils should be performed in accordance with ASTM D-3080 (Standard Test Method for Direct Shear Test of Soils under Consolidated Drained Conditions).

- Slake-Durability tests, if performed on materials obtained from the waste rock piles, should be performed in accordance with ASTM D-5312 (Standard Test Method for Slake Durability of Shales and Similar Weak Rocks).

- Freeze-Thaw tests, if performed on materials obtained from the waste rock piles, should be performed in accordance with ASTM D-4644 (Standard Test Method for Evaluation of Durability of Rock for Erosion Control under Freeze-Thaw Conditions).

- Organic content of soils should be performed in accordance with ASTM D-2974 (Standard Test Methods for Moisture, Ash, and Organic Matter of Peat and Other Organic Soils).

- Monitoring wells will be installed in accordance with ASTM 5092 (Design and Installation of Ground Water Monitoring Wells in Aquifers).

- Vibrating wire piezometers will be installed in accordance with USBR or U.S. Army Corps of Engineers requirements.

- Borehole inclinometers will be installed and monitored in accordance with ASTM D-6230 (Test Method for Monitoring Ground Movement Using Probe-Type Inclinometers).

- Monitoring wells will be protected in accordance with ASTM D-5787 (Standard Practice for Monitoring Well Protection).

- Groundwater conditions in the Surface Mine and Waste Rock Pile Areas should be evaluated in accordance with ASTM D-5979 (Standard Guide for Conceptualization and Characterization of Ground-Water Systems).

5.6 Quality Control

Quality control will be performed on a continuous basis by site personnel as work progress in the field. Field record books will be maintained as necessary and field logs will be maintained and copied daily to eliminate the possibility of lost data. Approximately 5 to 10 percent additional samples will be collected in the field, beyond those specified, for later testing if test results appear to be in error.

Samples will be handled, packaged, labeled and shipped to the testing laboratory in accordance with accepted ASTM and EPA standards. All testing by the laboratory will be performed in accordance with accepted ASTM standards including all required data and information reporting required by the standards.

Field logs of borings and test pits will be reviewed and corrected as necessary based on the laboratory testing. The geotechnical report will be developed by consultants for W.R. Grace and reviewed by the various parties involved in the program.

Surveying for location and elevation of borings and test pits will be performed in accordance with accepted survey standards of the American Congress on Surveying and Mapping (ACSM) and the National Society of Professional Surveyor (NSPS).

Table 5-2: Summary of Sampling Design

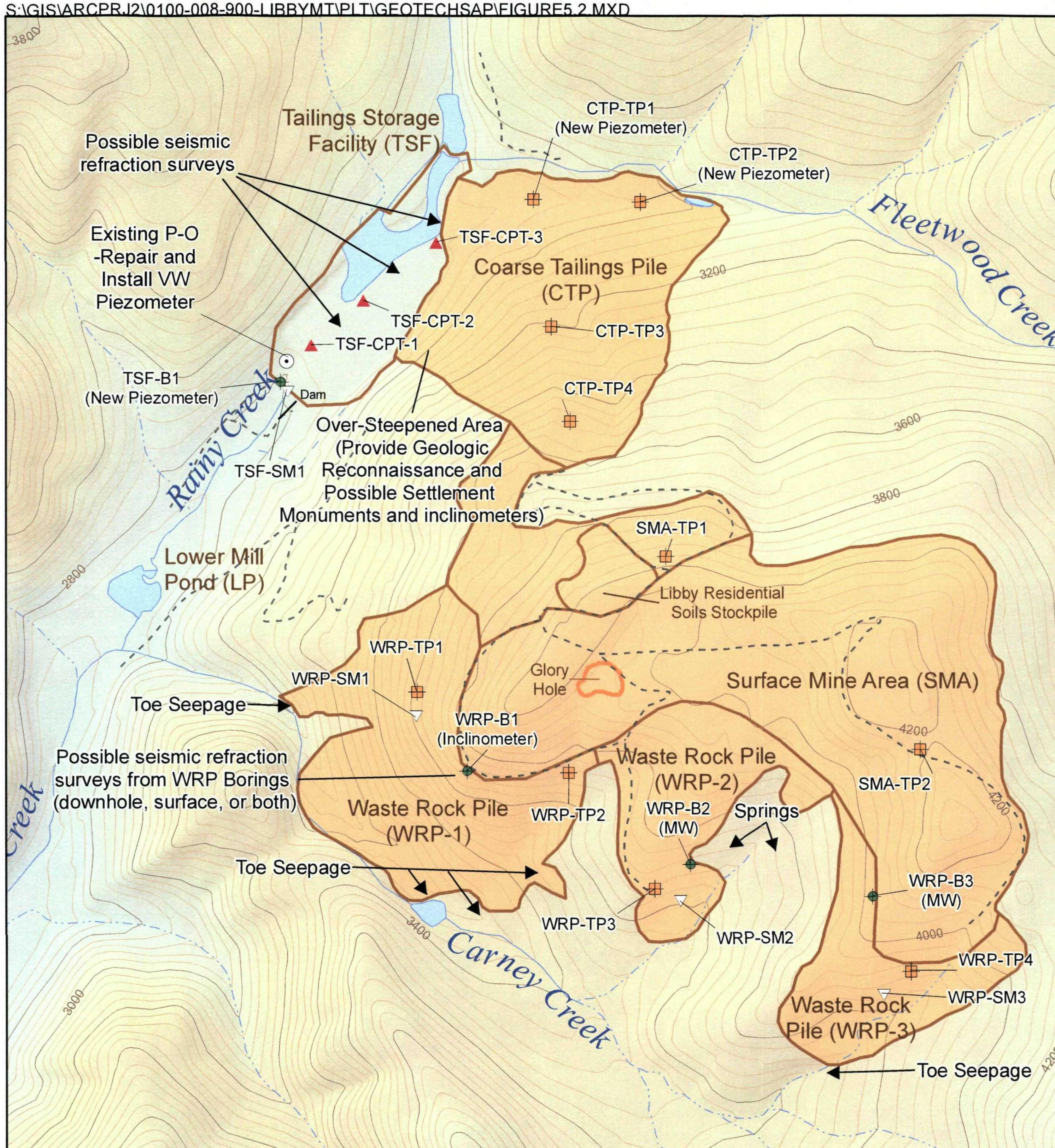
Boring, Test Pit or Item ID	Bulk Samples	Undisturbed Samples	Index Tests	Moisture-Density Tests	Compaction Tests	Strength Tests	Rock Durability Tests	Organic Content Tests	Piezometers or Monitoring Well	Comment
TSF-B1	2-3	2	3-4	2-3	1	1 TX	1-2 RQDs		Install New Piezo. & Transducer & Data Logger	At Max. Dam Section
TSF CPT-1 to 3	-----	-------	------	------	------	-------	-------	------	------	Std. CPT Rpt
TSF Existing Piezo. P-0	------	------	------	------	-------	------	-------	------	Install VW Piezo.	Repair Piezo.;Add Data Logger
Existing P-2 and PM-2									Install Transducers	Data Loggers
CTP-TP1	1-2	1	1-2	1	--------	------	-------	1	Install New Piezo.	----
CTP-TP2	1-2	1	1-2	1	-------	------	-----	1	Install New Piezo.	----
CTP-TP4	1-2	1	1	1	------	------	------	1	-------	-----
CTP Geologic Recon.	------	------	------	------	------	-----	------	-----	-------	Possible SM/Inclin
SMA-TP2	1	-	1				-------	1	-------	-------
WRP-B1	2-3	2-3	2-3	1-2	1	1-TX	------	------	--------	Install Inclinometer
WRP-B2	1-2	1-2	1-2	1-2	-----	------	------	------	-------	New MW
WRP-B3	1-2	1-2	1-2	1-2	------	-------	--------		--------	New MW
WRP-TP1	1-2	1	1	1	-------	------	1 F-T or S-D	1	-------	---
WRP-TP3	1-2	1	1	1	-----	------		1		---
WRP-TP4	1-2	1	1	1	1	1-DS	1 S-D or F-T	-----	------	----

Notes:
TSF denotes Tailing Storage Facility
CPT denotes Cone Penetrometer Test
F-T denotes Freeze-Thaw Test
CTP denotes Coarse Tailing Pile
WRP denotes Waste Rock Pile
SMA denotes Surface Mine Area
DS denotes Direct Shear Test.
VW denotes Vibrating Wire Piezometers
S-D denotes Slake-Durability Test
TX denotes Triaxial Shear Test

Settlement Monuments at TSF and WRP areas not shown and existing MWs not indicated although water level measurements required from all existing MWs

Table 5-2: Summary of Sampling Design

S:\GIS\ARCPRJ2\0100-008-900-LIBBYMT\PLT\GEOTECHSAP\FIGURE5.2.MXD



Legend

- MineRoads
- Glory Hole Area
- Mine Disturbance Areas
- Mine Disturbance
- Proposed New Borings (4)
- Proposed Cone Penetrometer Test (3)
- Proposed Test Pits (10)
- P-O Existing Non-Functional Piezometer
- SM-Proposed Settlement Monuments (4)
- MW New Groundwater Monitor Wells

Note: Locations of borings and test pits are approximate and may vary in the field depending on access and field conditions encountered.

0 1,500 3,000 Feet N

LIBBY MONTANA SUPERFUND SITE
OPERABLE UNIT 3

FIGURE 5-2

PHASE III SAP GEOTECHNICAL EXPLORATIONS FOR TAILINGS & MINE AREA

DATE: JUL 22, 2010		FORMATION ENVIRONMENTAL
BY: SLG	FOR: JHR	